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PROCEEDINGS

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VOL. 70 JANUARY, 1944 No. 1

TECHNICAL PAPERS

AND

DISCUSSIONS

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CURRENT PAPERS AND DISCUSSIONS

		Discussion closes
The Hydraulic Jump in Sloping Channels. *Carl E. Kindsvater*	Nov., 1942	
Discussion	Feb., May, June, Sept., 1943	Closed*
Application of Soil Mechanics in Designing Building Foundations. *A. Casagrande and R. E. Fadum*	Nov., 1942	
Discussion	Mar., June, Sept., 1943	Closed*
Flow Characteristics at Rectangular Open-Channel Junctions. *Edward H. Taylor*	Nov., 1942	
Discussion	June, Sept., Oct., 1943, Jan., 1944	Closed
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Discussion	Dec., 1942, Mar., Apr., June, Sept., 1943	Closed*
Concrete Reservoirs of the Vertical-Beam Type. *C. Maxwell Stanley*	Dec., 1942	
Discussion	Mar., Apr., May, 1943	Closed*
Illinois Water Litigation, 1940–1941. *Langdon Pearse*	Dec., 1942	
Discussion	May, June, 1943	Closed*
Organizing and Financing Sewage Treatment Projects. *Samuel A. Greeley*	Dec., 1942	
Discussion	Dec., 1942, Apr., May, Sept., 1943, Jan., 1944	Closed
Pendleton Levee Failure. *Kenneth E. Fields and William L. Wells*	Dec., 1942	
Discussion	Dec., 1942, June, 1943	Closed*
Dewatering, Incineration, and Use of Sewage Sludge: A Symposium	Jan., 1943	
Discussion	Feb., June, Sept., 1943	Closed*
Primary Rôle of Meteorology in Flood Flow Estimating. *Merrill Bernard*	Jan., 1943	
Discussion	Mar., Apr., June, Oct., Nov., 1943	Closed*
Determination of Kutter's *n* for Sewers Partly Filled. *C. Frank Johnson*	Feb., 1943	
Discussion	Apr., Sept., 1943, Jan., 1944	Closed
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Discussion	Apr., June, Nov., Dec., 1943, Jan., 1944	Closed*
Flow Around Bends in Stable Channels. *C. A. Mockmore*	Mar., 1943	
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Highway Engineering Education: Progress Report of the Committee of the Highway Division	Apr., 1943	
Effect of Turbulence on Sedimentation. *William E. Dobbins*	Feb., 1943	
Discussion	Apr., May, June, Oct., 1943, Jan., 1944	Feb. 1, 1944
Simplified Analysis of Skewed Reinforced Concrete Frames and Arches. *Richard M. Hodges*	May, 1943	
Discussion	Sept., Nov., 1943	Feb. 1, 1944
A Symmetrically Loaded Base Slab on an Elastic Foundation. *Stanley U. Benscoter*	May, 1943	
Discussion	Sept., Nov., 1943	Feb. 1, 1944
A Method of Computing Urban Runoff. *W. I. Hicks*	Apr., 1943	
Discussion	Dec., 1943	Feb. 1, 1944
Rôle of the Land During Flood Periods. *W. W. Horner*	May, 1943	
Discussion	June, Sept., Oct., Dec., 1943	Feb. 1, 1944
Sedimentation in Reservoirs. *Berard J. Witzig*	June, 1943	
Discussion	Sept., Oct., Nov., Dec., 1943	Feb. 1, 1944
Development of the Chicago Type Bascule Bridge. *Donald N. Becker*	Feb., 1943	
Discussion	June, Sept., 1943, Jan., 1944	Feb. 1, 1944
Strength of Slender Beams. *George Winter*	June, 1943	
Discussion	Sept., Dec., 1943	Feb. 1, 1944
Sludge Drying Developments at Chicago, Ill. *Lloyd M. Johnson*	Sept., 1943	
Discussion	Sept., Oct., 1943	Feb. 1, 1944
Rainbow Arch Bridge Over Niagara Gorge: A Symposium	Oct., 1943	
Discussion	Dec., 1943, Jan., 1944	Mar. 1, 1944
Storage and the Unit Hydrograph. *C. O. Clark*	Nov., 1943	Apr. 1, 1944
Rigidity and Aerodynamic Stability of Suspension Bridges. *D. B. Steinman*	Nov., 1943	Apr. 1, 1944
Unusual Cutoff Problems—Dams of the Tennessee Valley Authority: A Symposium	Nov., 1943	Apr. 1, 1944
Shear Effect on the Strength of Struts. *S. Sergev*	Nov., 1943	Apr. 1, 1944
Geology in Highway Engineering. *Marshall T. Huntting*	Dec., 1943	May 1, 1944
Original and Readjusted Repayment Contracts, Boulder Canyon Project. *Clay C. Elder*	Dec., 1943	
Discussion	Jan., 1944	May 1, 1944
Failure of the Tacoma Narrows Bridge: Report of the Special Committee of the Board of Direction	Dec., 1943	May 1, 1944

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* Publication of closing discussion pending.

CONTENTS FOR JANUARY, 1944

PAPERS

PAGE

Surges in Panama Canal Reproduced in Model.
By F. W. Edwards and Edward Soucek 3

Coefficients for Velocity Distribution in Open-Channel Flow.
By William S. Eisenlohr, Jr. 15

Military Airfields: A Symposium 27

DISCUSSIONS

Organizing and Financing Sewage Treatment Projects.
By Samuel A. Greeley 91

Determination of Kutter's *n* for Sewers Partly Filled.
By C. Frank Johnson 93

Characteristics of Heavy Rainfall in New Mexico and Arizona.
By Walter B. Langbein, and Emil P. Schuleen 95

Effect of Turbulence on Sedimentation.
By E. R. Van Driest 103

Development of the Chicago Type Bascule Bridge.
By C. B. McCullough 105

Rainbow Arch Bridge Over Niagara Gorge: A Symposium.
By C. M. Goodrich, and T. Kennard Thomson 114

CONTENTS FOR JANUARY, 1944 (*Continued*)

PAGE

Flow Characteristics at Rectangular Open-Channel Junctions.
By Edward H. Taylor 119

Original and Readjusted Repayment Contracts, Boulder Canyon Project.
By W. L. Chadwick 122

For Index to all Papers, the discussion of which is current in PROCEEDINGS, *see page 2*

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

SURGES IN PANAMA CANAL REPRODUCED IN MODEL

By F. W. Edwards,[1] M. Am. Soc. C. E., and Edward Soucek,[2] Assoc. M. Am. Soc. C. E.

Synopsis

A 1 : 60 scale model of Gaillard (formerly Culebra) Cut was built in the Panama Canal Hydraulics Laboratory for the study of surges caused by lock operations. The verification tests described in this paper consist of water-surface observations of model and prototype at corresponding locations and times, during and after corresponding lock operations. The success of these tests was made possible by the cooperation of various divisions of the Panama Canal.

The Problem

Gaillard Cut is an 8¾-mile reach of the summit level of the Panama Canal, extending from the Pedro Miguel Lock to Gatun Lake at Gamboa. Fig. 1 shows cross sections at the observation stations. The operation of filling Pedro Miguel Lock creates surges which increase the difficulty of handling ships in the Cut. Although the condition is not particularly serious at present (1942), it will be aggravated by the operation of a proposed larger lock. The navigation conditions at the junction of the proposed by-pass channel for the proposed third lock and the existing channel are of special importance. No further consideration will be given to the navigation problems involved. This paper is confined to a presentation of the results of verification tests.

Prototype Tests

The prototype observations were made in December, 1939, prior to the construction of the model. All tests were conducted at night after traffic in the Cut had ceased and the water surface had become quiescent.

Pedro Miguel Lock consists of twin chambers separated by a center wall. There are three filling culverts, one in each side wall and one in the center wall. The center-wall culvert is valved in such a manner that it can be opened to

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by June 1, 1944.

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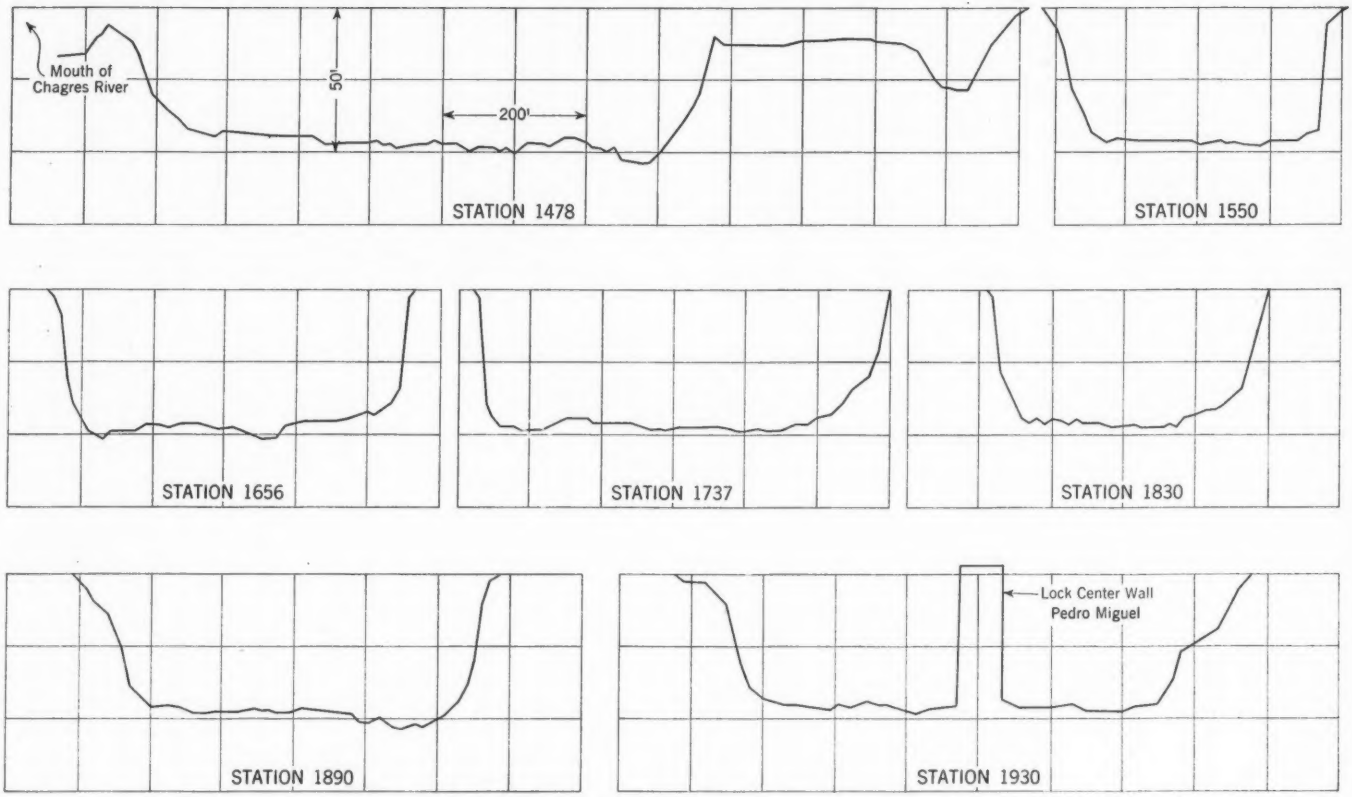


FIG. 1.—CHANNEL CROSS SECTIONS, VIEWED TOWARD PEDRO MIGUEL LOCK

either chamber or to both simultaneously. The schedule of lock operations used in the tests was as follows:

Test No.	Description
I	A two-culvert fill of the East Chamber
II	A three-culvert fill of both chambers
III	Two three-culvert fills of both chambers; interval between fills 88 min
IV	Repetition of Test No. III
V	Two three-culvert fills of both chambers; interval between fills 44 min

It was necessary to plan the tests carefully because there was no means of communication with the observers. Observers met at Pedro Miguel Lock at 11:30 p. m. each night, received special instructions regarding the test scheduled for that night, were issued equipment and supplies, and were transported to their stations by launch. Synchronized stop-watches, previously checked for accuracy, were used for timing all readings and operations. All watches were started simultaneously, wound every hour, and stopped simultaneously for checking after the observers had reassembled. No adjustment of observations for errors in timing was found necessary. All equipment, such as watches, data sheets, flashlights, etc., was furnished in duplicate to each party. A party, consisting of two observers, was located at each station and at the lock control house.

Prior to the tests, a series of water-level observations was made at 10-sec intervals at one of the stations. This series was made in the daytime, under normal traffic conditions, to find the interval for test readings which would define the surges adequately. A 1-min interval was found satisfactory and this was adopted for the tests. The duration of observations each night averaged more than 5 hr.

With the exception of permanent gages at Pedro Miguel and Gamboa, temporary staff gages were installed for the verification tests. The gages were graduated in tenths of feet but readings were estimated to hundredths. The gages were leveled, prior to the tests, on a quiescent water surface during early morning hours before traffic started through the Cut. A check upon the accuracy of gage settings was obtained each night by taking readings for several minutes prior to lock operation. Small adjustments were made on the basis of these observations. Various methods of cross-checking the gage elevations were used and all indicated that the adjusted elevations were correct to the nearest one-hundredth of a foot.

The Model

The model size was selected on the basis of economy, sensitivity, and relative freedom from surface tension. A distorted model was considered but rejected. Although friction was relatively unimportant, the concrete surfaces in contact with water were finished so as to give proportional friction losses in model and prototype. Lock hydraulic systems were not simulated, but filling

Fig. 1.—Channel Cross Sections, Viewed Toward Pedro Miguel Locks

curves—obtained during prototype tests—were approximated in the model by the use of manually operated gates. The feature most difficult to simulate was the junction of the Cut with Gatun Lake. An uncontrolled forebay large enough to simulate the lake was impracticable. A relatively small forebay (684 sq ft), with overflow troughs to maintain a constant level, was adopted. The overflow length and rate of inflow to the forebay required for similitude were determined by trial. Any constant inflow within a considerable range could be used without disturbing the similitude. This characteristic of the model supports the confidence which the writers have in the ability of the model to indicate surge conditions for which no prototype comparison is possible.

Water-surface observations in the model were made on inclined staff gages which were designed so as to minimize local disturbances. The gages were graduated at vertical intervals of one-sixtieth of a foot, representing one foot on the prototype. Readings were estimated to the nearest tenth of a foot, prototype dimension. This method of observation may be considered crude, but has been entirely justified by the results obtained. Other proposed methods had important disadvantages.

Model observations were timed by the use of a single electric timer, located at a control station. A buzzer system was used to signal the observers when readings were desired.

Nature of the Surges

Test No. II will be used to illustrate the surge phenomenon. Fig. 2 shows water-surface profiles at 5-min intervals following the beginning of lock filling. Although these profiles are based on actual prototype observations, the small number of stations permits considerable latitude in interpretation. For this reason, the profiles should be considered pictorial rather than exact.

Considerable variation in section and in average depth exists in the Cut as is apparent from Fig. 1. From consideration of approximately twenty cross sections, the average depth, y, was estimated as 41.0 ft. The corresponding wave velocity computed by the formula

$$V = \sqrt{g\,y} \qquad (1)$$

is 36.4 ft per sec. Velocities of flow are so low that wave velocities may be considered to be absolute. The point of the arrow shown with each profile of Fig. 2 indicates the position which the front of the wave would occupy if it traveled along the Cut at a uniform velocity of 36.4 ft per sec. Fair agreement between calculated and observed positions exists even after 100 min. Friction, variation in channel characteristics, and uncertainty regarding the exact stations from which reflection should be considered to occur, especially at the lake end of the Cut, would be expected to produce some discrepancy.

The surface width near the lock is about 750 ft. The rate of inflow to the locks varies, but the maximum rate in the test under consideration was approximately 22,500 cu ft per sec. From discharge, wave velocity, and surface width, the drawdown calculated by continuity is about 0.8 ft, substantially the depth which occurred. Some sections of the Cut are much narrower, the

average top width being only about 480 ft. However, no increase in surge height occurred in the narrower portions of the channel. The surge height is not to be confused with velocity head.

In the 5-min profile (Fig. 2), the water at the initial level is still at rest, while flow occurs toward the lock in the depressed reach. Five minutes later, inflow to the lock has ceased and water is beginning to pile up against the lock.

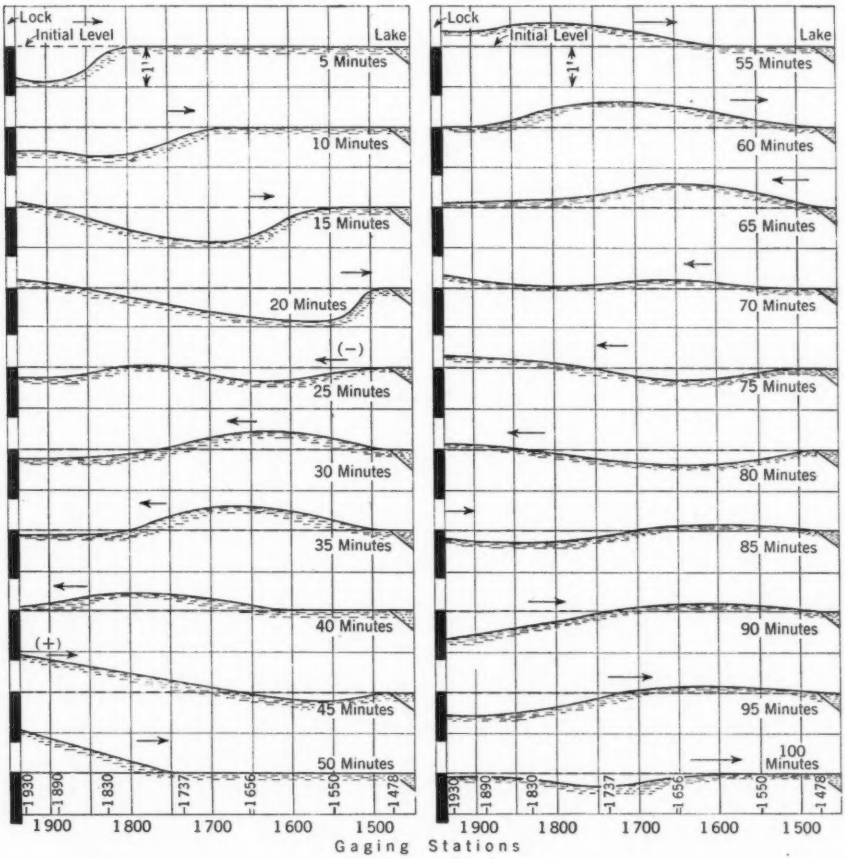
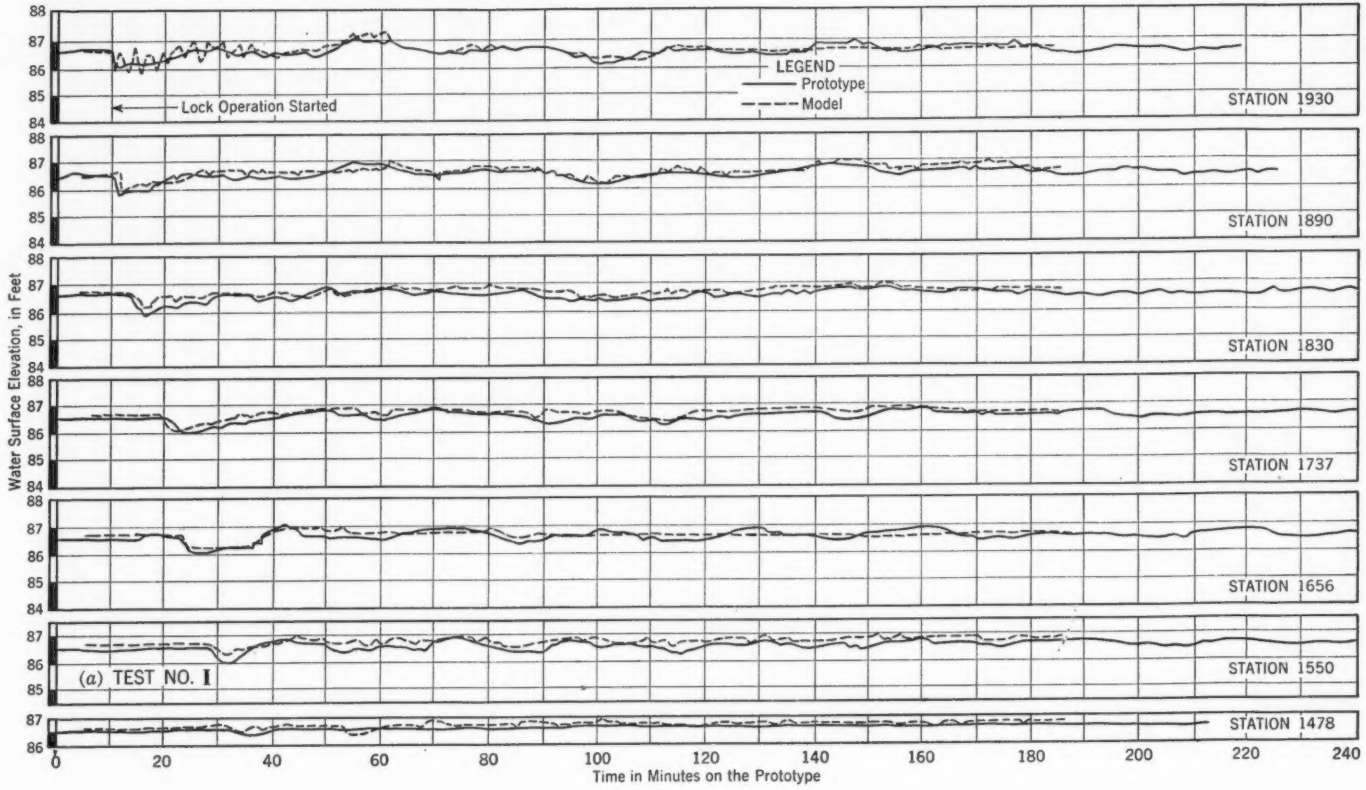


FIG. 2.—WATER-SURFACE PROFILES AT 5-MIN INTERVALS

In the 15-min profile, the surface at the lock has returned to its initial level. The "sag" in the water level at this time is a measure of the volume drawn into the locks—about 8,000,000 cu ft. Water is at rest both ahead of and behind the "sag," but it is in motion toward the lock under the "sag." Water is being transferred past the moving "sag" at a rate approximately equal to the maximum rate of inflow to the locks which occurred nearly 15 min earlier. The difference in slopes at the front and back of the "sag" is indicative of the


Water Surface Elevation, in Feet
LEGEND
Prototype
Model
Lock Operation Started
STATION 1930
STATION 1890
STATION 1830
STATION 1737
STATION 1656
STATION 1550
STATION 1478
(a) TEST NO. I
Time in Minutes on the Prototype

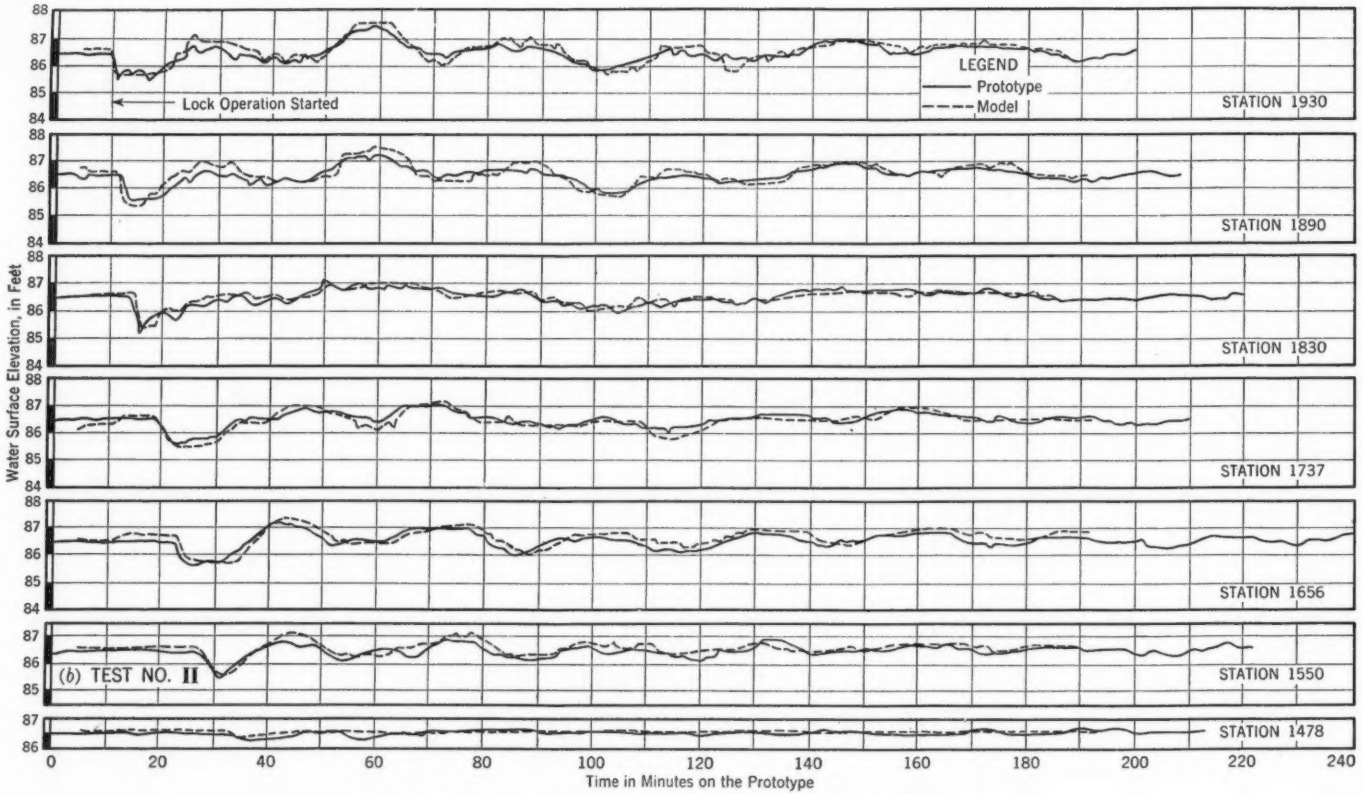
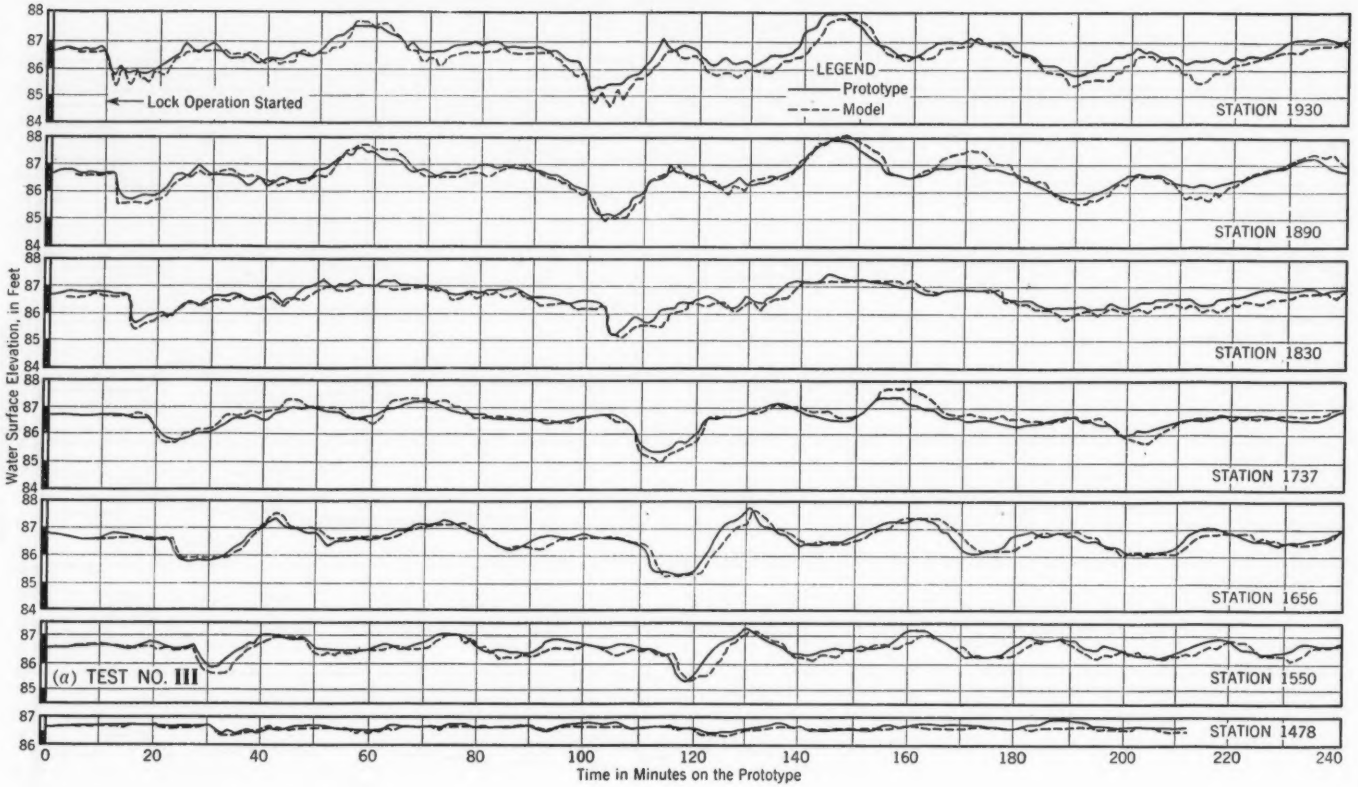


FIG. 3.—COMPARISON OF WATER-SURFACE PROFILES, MODEL AND PROTOTYPE


Water Surface Elevation, in Feet
LEGEND
Prototype
Model
Lock Operation Started
STATION 1930
STATION 1890
STATION 1830
STATION 1737
STATION 1656
STATION 1550
STATION 1478
(a) TEST NO. III
Time in Minutes on the Prototype

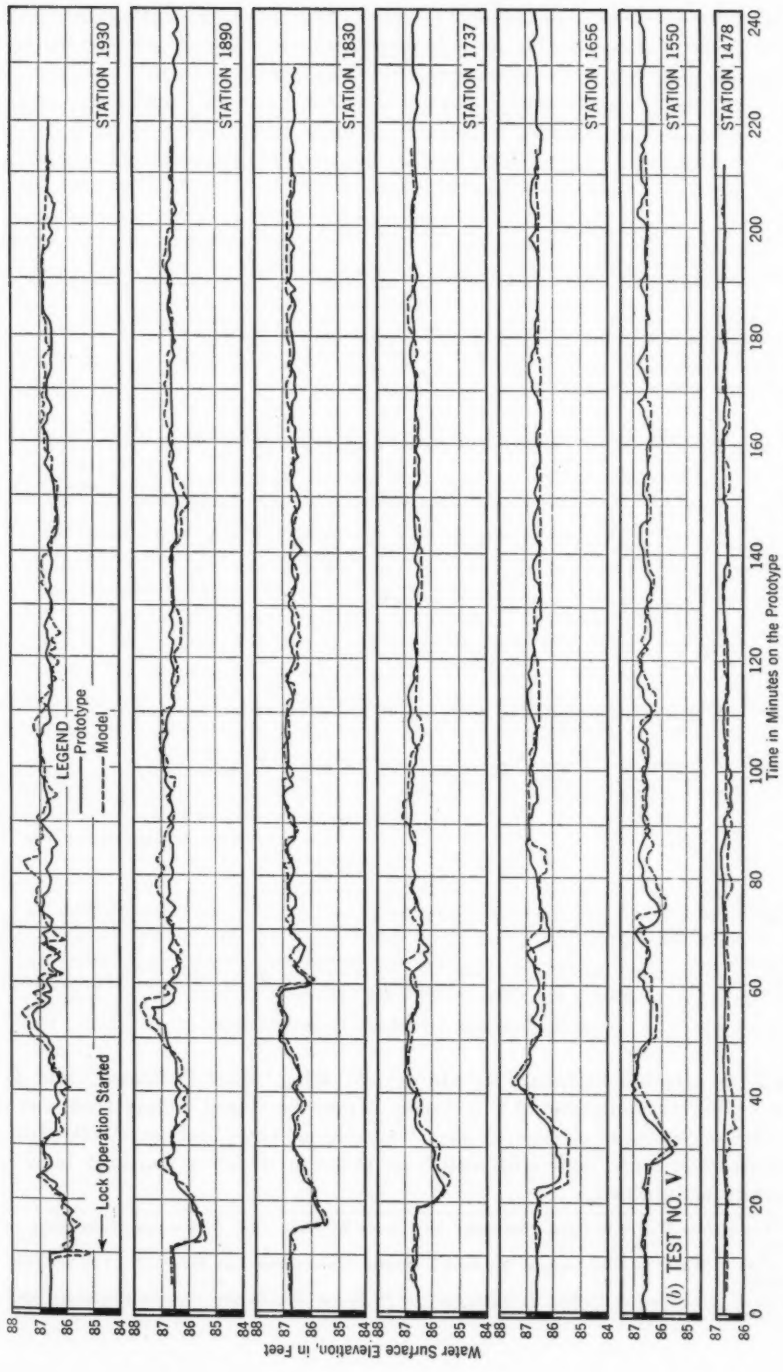


FIG. 4.—COMPARISON OF WATER-SURFACE PROFILES, MODEL AND PROTOTYPE

variation in rate of flow during the lock filling period. The length of the "sag" is a measure of the duration of inflow to the lock. The 20-min profile is a further development of the condition which has been explained.

A rigorous treatment would require that the front and back faces of the "sag" be treated separately. However, no harm to the description, and considerable simplification, result if the disturbance is considered from now on as a single wave. For discussions of the mechanics of wave reflection, reference

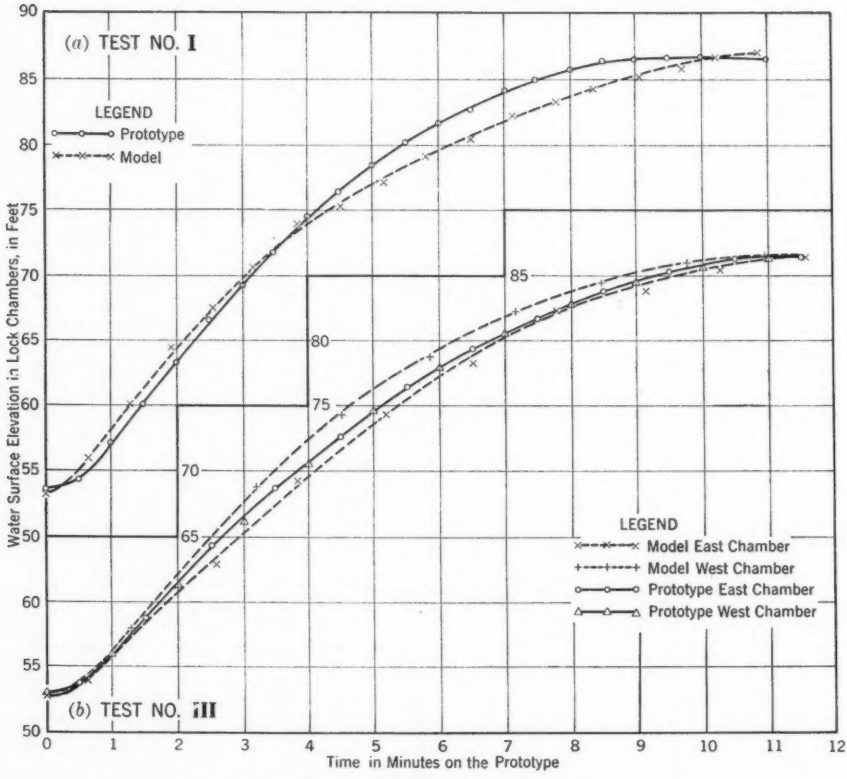


FIG. 5.—COMPARISON OF LOCK FILLING CURVES

is made to selected literature by Horace W. King,[3] Earl I. Brown,[4] and H. Rouse.[5] For the purposes of this paper, it may be stated simply that waves are reflected without change in sign from closed ends and negatively from junctions with large bodies of water, or those in which a constant level is artificially maintained.

[3] "Translatory Waves in Open Channels," by Horace W. King, *Civil Engineering*, June, 1933, pp. 319–321.

[4] "Flow of Water in Tidal Canals," by Earl I. Brown, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), pp. 749–834.

[5] "Fluid Mechanics for Hydraulic Engineers," by H. Rouse, *Engineering Societies Monograph*, 1938, pp. 353–402.

The 25-min profile (Fig. 2) is a stage of the reflection in the negative sense and the 30-min, 35-min, and 40-min profiles show the return of a somewhat attenuated "hump" from the lake toward the lock. The 45-min and 50-min profiles show the positive reflection at the lock. The 55-min, 60-min, and 65-min profiles show the travel of the reflected "hump" from the lock toward the lake. The 70-min profile is the second occurrence of negative reflection and the 75-min, 80-min, and 85-min profiles show the passage of a now considerably attenuated "sag" toward the lock. The 90-min profile is the second occurrence of positive reflection. The 95-min and 100-min profiles, comparable approximately to the 5-min and 10-min profiles, show the beginning of another cycle.

The effect of successive lock fillings during either of the positive reflection phases, exemplified by Tests Nos. III and V, is predictable, at least qualitatively.

Comparison of Model and Prototype

The conformity of model observations with prototype Tests Nos. I, II, III, and V is shown in Figs. 3 and 4. Test No. IV, which was a repetition of Test No. III, is not reproduced here as the results were identical. The agreement is considered to be very satisfactory. It is evident that conclusions based upon the model observations would be the same as those based upon the prototype tests. Refinements could have been adopted which would have produced better results, but they were not warranted for the studies undertaken. It is believed that more accurate reproduction of the lock filling curves would have eliminated almost entirely the relatively small discrepancies which are indicated. Comparisons of typical filling curves are shown in Fig. 5.

A comparison of Figs. 4(*a*) and 4(*b*) is of special interest because it illustrates the result of synchronization and of anti-synchronization of the effects of consecutive lock operations.

Acknowledgments

The success of these tests was made possible by the cooperation of various divisions of the Panama Canal. Maj.-Gen. Glen E. Edgerton, M. Am. Soc. C. E., is Governor of the Canal Zone. At the time these tests were conducted Maj.-Gen. T. B. Larkin, M. Am. Soc. C. E., was supervising engineer on the Third Locks project and E. S. Randolph, M. Am. Soc. C. E., was designing engineer.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

COEFFICIENTS FOR VELOCITY DISTRIBUTION IN OPEN-CHANNEL FLOW

By William S. Eisenlohr, Jr.,[1] Assoc. M. Am. Soc. C. E.

Synopsis

Where it is necessary to take into consideration the nonuniform distribution of velocity of water flowing in open channels, it is customary to use the velocity head based on the mean velocity and a coefficient. This coefficient is fixed by the equation selected: It is the ratio of either the mean of the squares of the local velocities to the square of the mean, or the mean of the cubes of the local velocities to the cube of the mean velocity. The Chézy and Manning equations are frequently considered to be energy equations but this paper shows that, from the nature of their coefficients, they are truly momentum equations. They will not yield practical solutions when considered as energy equations except for uniform velocity distribution. The proper coefficient to be used in momentum equations is then shown to be C_m, the ratio of the mean of the squares of the local velocities to the square of the mean velocity.

Introduction

In the practical solution of most problems involving a stream of water flowing in a pipe or open channel it is not necessary to consider the variation of velocity throughout the cross section; use of the average velocity will give sufficiently accurate results. Textbooks on hydraulics, therefore, merely develop the equations of flow through an elemental stream tube and the resulting equations are then used for total flows. There are problems in nonuniform flow, however, in which a substantial part of the total change in momentum and kinetic energy between two cross sections may be due to a change in the distribution of velocity as distinguished from a change in the mean velocity. In such problems the nonuniform distribution of velocity must be taken into account by integrating, over the cross section, the equation of flow through an elemental stream tube.

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **June 1, 1944.**

[1] Associate Hydr. Engr., Water Resources Branch, U. S. Geological Survey, Washington, D. C.

The momentum and kinetic energy of a stream of water are functions of the square and cube of the velocity, respectively. Therefore, the total momentum and kinetic energy possessed by a stream at any cross section are the sums of the momentums and energies of the flow in the different parts of the cross section. Practically, the same result can be obtained by using the average velocity and a coefficient which has been evaluated by the integration process for the particular velocity distribution being studied. These coefficients are always greater than unity. They appear to be so related that for ordinary use the coefficient of momentum[2] can be taken as

$$C_m = 1 + \frac{\epsilon^2}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

and the coefficient of energy as

$$C_e = 1 + \epsilon^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1b)$$

in which $\epsilon = \frac{u_{\max}}{V} - 1$. Some confusion seems to exist as to which coefficient should be used, as it is difficult to appraise correctly the factors involved in such a choice if one is accustomed to the use of formulas based on the mean velocity alone. The purpose of this paper is to explain the various factors involved and show their proper use. The usual desire is to apply some "correction coefficient" to the ordinary formula and let that suffice. Although the use of a proper coefficient is the practical solution, it should be realized that such a coefficient is the result of integrating the fundamental hydraulic equation involved, and is in no sense a "correction." To understand properly the various coefficients it will be necessary to consider the fundamental principles involved, and to develop the equations for integration. This can be done best by an illustrative example for which purpose the "Bernoulli" equation has been chosen. The numerical values of such coefficients are usually obtained from observations of the velocity distribution in the flow problem being studied or from some characteristic of flow which will yield a reasonable approximation of the velocity distribution.

Notation

The following letter symbols, used in this paper, conform essentially to American Standard Letter Symbols for Hydraulics,[3] prepared by a Committee of the American Standards Association, with Society representation, and approved by the Association in 1942. Subscripts denoting particular values are given at the end.

A = area, total cross sectional;
a = acceleration, linear;
B = bulk or volume (see Eq. 14*a*);
C = coefficient of flow (Chézy) in $V = C\sqrt{RS}$;
C_A, C_B = coefficients of force distribution in the energy equation;

[2] "Die Bestimmung der Lage der Energielinie bei fliessenden Gewässern mit Hilfe des Geschwindigkeitshöhen-Ausgleichwertes," by Th. Rehbock, *Der Bauingenieur*, No. 15, Vol. 3, August 15, 1922, pp. 453–455.

[3] ASA—Z10.2—1942.

C_e = coefficient of energy distribution = $\frac{\overline{u^3}}{V^3}$;

C_m = coefficient of momentum distribution = $\frac{\overline{u^2}}{V^2}$;

F = force;

f = Darcy coefficient of friction = 4λ;

g = gravitational acceleration;

h_f = head, lost by friction;

k = a constant ratio of pressure intensities (see Eqs. 4);

L = length; length of stream tube or channel between sections;

m = mass;

n = coefficient of roughness in the Manning equation;

P = wetted perimeter;

p = pressure per unit area;

Q = discharge (rate of flow);

R = hydraulic radius;

S = friction slope; slope of momentum grade line;

t = time;

u = velocity component in the direction of flow; velocity of flow through a stream tube;

V = mean velocity, $\frac{Q}{A}$;

Z = elevation, above datum;

γ = specific weight;

ϵ = velocity distribution constant = $\frac{u_{max}}{V} - 1$;

θ = angle between axis of a stream tube and the horizontal, $\sin^{-1}\left(\frac{Z_A - Z_B}{L}\right)$;

λ = resistance coefficient = $\frac{f}{4}$; the ratio of boundary shear to kinetic energy based on the mean velocity;

ρ = density, mass per unit volume = $\frac{\gamma}{g}$;

τ_0 = unit shear at the channel boundary.

Subscripts and Special Characters.—Letter subscripts refer a symbol to various conditions as follows:

A = "section A" (Fig. 1);

B = "section B" (Fig. 1);

A-B = "between section A and section B";

e = "energy equation";

m = "momentum equation";

n = "normal to the sides of a stream tube" (see Eqs. 4);

p = "pressure";

r = "resisting";

u = "uniform velocity distribution."

A bar over a symbol signifies the average value of the quantity indicated. Subscripts 1 and 2 refer a symbol to stream tubes 1 and 2 (Figs. 1, 2), respectively.

Momentum vs. Energy

Standard textbooks on hydraulics usually demonstrate the proof of the Bernoulli equation either by the momentum theorem or by the energy principle, although they are not always so labeled. In either case it is usual to assume that the stream is composed of a bundle of "stream tubes" of differential area in cross section. A stream tube may be defined as a small tube bounded by a surface which at every point is tangent to the velocity vector at that point. There is no flow across the bounding surface of a stream tube. (A more correct definition for turbulent flow is that the temporal average of flow across a stream tube boundary is zero.) Flow through the stream tube is then similar to flow through a small pipe of varying cross section. The flow through a single stream tube may be analyzed and the resulting differential equation integrated over the cross section. The basic equation by the momentum theorem or energy principle is that of Newton's second law of motion, $F = m\,a$, wherein the substitution $a = \frac{u^2_B - u^2_A}{2\,L}$ has been made. The difference between the two methods lies in which side of the equation the distance L is placed. If it is placed on the right, the momentum equation becomes:

$$F = m\,\frac{u^2_B - u^2_A}{2\,L} \qquad (2)$$

in the form of momentum change per unit time—that is, $\frac{F\,t}{t} = \frac{m\,u}{t}$; and $F = \frac{m\,u}{t}\left(\times\frac{L}{L}\right) = \frac{m\,u^2}{L}$. The change of the momentum with time is equal to the resultant force acting on the mass.

If the distance L is placed on the left the energy relation is

$$F\,L = \tfrac{1}{2}\,m\,(u^2_B - u^2_A) \qquad (3a)$$

which states that the change in kinetic energy is the result of work done by the impressed forces. If the change in unit time is considered, as was done for momentum, and Eq. 3*a* is divided by $t = 1$:

$$F\,u = \tfrac{1}{2}\,m\,[u^2_B - u^2_A]_{t=1} \qquad (3b)$$

To apply these equations it is necessary, of course, to have steady flow, or flow which on the average may be regarded as steady. Only under such a condition would the system be in equilibrium.

It is most important to realize the significance of the method of considering the distance L as used in the preceding paragraph. In the momentum equation, L is the length of the stream tube in which the change in momentum takes place. It may have any value desired but it is the same for every stream tube within the stream. In the momentum equation the forces are considered to be

acting independent of any motion. On the other hand, in the energy equation the forces must act through a specified distance (u, if Eq. 3*b* is used). Since the velocity distribution is nonuniform, u is different for each stream tube. This makes an extremely complicated differential equation, the integration of which does not yield a solution suitable for practical use. To simplify the problem, most textbook writers assume a uniform distribution of velocity when deriving the Bernoulli equation by means of the energy principle. Using this assumption, u is the same for each stream tube. The distances through which the forces act are thus the same for all stream tubes. The integration then can be performed in the same way as for the momentum equation and the result is an equation identical with that derived from the momentum theorem. An equation is thus derived by the energy principle, based on assumed uniform velocity distribution, giving the same answer as that derived by the momentum theorem. The result is an understandable confusion as to whether to use the coefficient of momentum distribution or the coefficient of energy distribution.

The Momentum Theorem

Many ideas frequently thought of in terms of the energy relation are really momentum concepts. The following paragraph is taken almost verbatim from the explanation given by L. Prandtl.[4]

The value of the momentum theorem is that it contains statements about conditions along the boundaries of a region only. The internal forces cancel by the principle of action and reaction, when summing over all the masses of the system, leaving only the external forces to be considered. Now the change in momentum of a mass of fluid which is bounded in any way, and is in steady motion, is due entirely to the fact that the boundaries of the fluid are displaced by the motion; every particle of the fluid in the interior is replaced by another which has taken up its velocity. What happens at the boundaries may be shown in the case of a stream tube. It is an essential feature of the momentum theorem that every individual mass belonging to the system shall remain in the system and that no new masses shall enter it. Thus the boundary surfaces drawn for the purpose of applying the theorem are not displaced with the flow; they are "fluid surfaces." Hence in the case of the stream tube there is a mass dm which leaves the downstream end of the stream tube in unit time. During the same time, by the law of continuity, an equal mass enters the upstream end of the stream tube. Thus, in unit time the stream tube contributes to the total change in momentum the positive momentum $dm\ u_B$ at the downstream end and at the same time the negative momentum $dm\ u_A$ is contributed at the upstream end. These changes in momentum per unit time may immediately be expressed as forces exerted from outside on the mass of fluid in question across the cross sections at the ends of the stream tube.

Development of the Momentum Equation

Proceeding now to develop the equation of flow according to the momentum theorem, consider the channel shown in Fig. 1. The stream can be considered

[4] "The Physics of Solids and Fluids with Recent Developments," Pt. 2, by L. Prandtl, 1936, p. 223.

to be divided into stream tubes of differential area dA. The change in momentum between sections A and B is the result of the external forces. The pressure force on the upstream end is $p_A\,dA_A$, and on the downstream end is $p_B\,dA_B$. Another basic assumption that must be made is that the flow is gradually varied. With this assumption the development can be simplified by letting $dA_A = dA_B = dA$ without restricting the variation in velocity.

(The validity of this assumption is easily shown. Let $p_A = p + \delta p$; $p_B = p$; $dA_A = dA$, and $dA_B = dA + \delta A$. In addition to the pressure forces on the ends of the stream tube, there is also a component in the direction of motion of the normal pressure p_n on the sides of the stream tube, which is equivalent

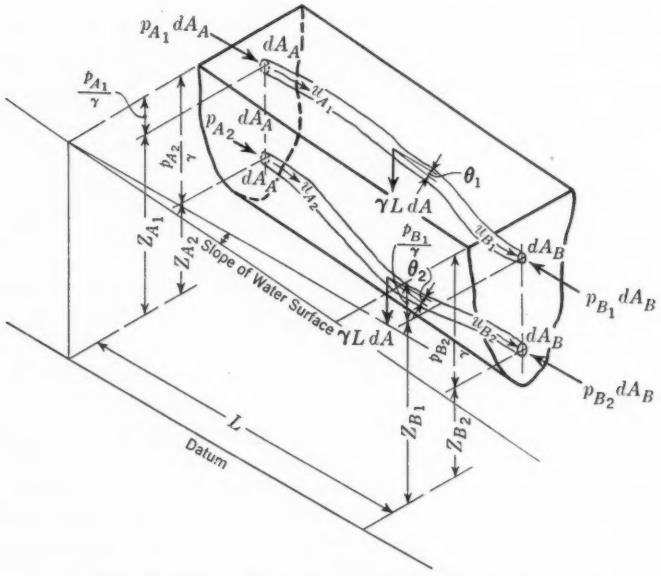


FIG. 1.—MOMENTUM DIAGRAM; ALSO ENERGY DIAGRAM AT $t = 0$

to $p_n\,\delta A$. Since p_n lies between p_A and p_B it can be expressed as $p + k\,\delta p$. The pressure forces are then:

$$F_p = p_A\,dA_A + p_n\,\delta A - p_B\,dA_B \ldots\ldots\ldots\ldots\ldots\ldots (4a)$$

$$F_p = (p + \delta p)\,dA + (p + k\,\delta p)\,\delta A - p\,(dA + \delta A) \ldots\ldots\ldots (4b)$$

and

$$F_p = \delta p\,dA + k\,\delta p\,\delta A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4c)$$

The first term of Eq. 4c is the equivalent of the resultant pressure force as obtained by assuming equal areas. The only error introduced in this assumption is the neglect of the term $k\,\delta p\,\delta A$ which, as the product of two differentials, is negligible.)

The resultant pressure force is then $(p_A - p_B)\,dA$. The gravity force is the component along the axis of the stream tube of the weight of the tube, $\gamma\,L\,dA$, which is $\gamma\,L\,dA \sin\theta$. In addition to the pressure and gravity forces,

there is the resisting force along the boundary of the stream tube. For the present this will be designated simply $dF_{r,m}$.

Inserting the foregoing quantities in Eq. 2:

$$p_A\,dA - p_B\,dA + \gamma\,L\,dA\sin\theta - dF_{r,m} = \rho\,L\,dA\,\frac{u^2_B - u^2_A}{2\,L} \quad \ldots\ldots (5)$$

substituting for $\sin\theta$ and rearranging:

$$dF_{r,m} = \left[\left(\frac{p_A}{\gamma} + Z_A\right)dA - \left(\frac{p_B}{\gamma} + Z_B\right)dA + \frac{u^2_A}{2\,g}\,dA - \frac{u^2_B}{2\,g}\,dA\right]\gamma \quad \ldots (6)$$

The total resisting force $F_{r,m}$ can now be obtained by integrating the right side of the equation with respect to dA. This is the important step. Since $\left(\frac{p}{\gamma} + Z\right)$ is the height of the water surface above the datum plane, it is the same for all stream tubes. The integration of the first two terms is easily performed. The last two terms cannot be integrated in their present forms.

The coefficient for nonuniform distribution of velocity is now introduced. The term C_m is defined as the coefficient of momentum distribution due to nonuniform distribution of velocity. It is expressed as:

$$C_m = \frac{\overline{u^2}}{V^2} \quad \text{or} \quad C_m = \frac{\int u^2\,dA}{V^2\,A}.$$

Integrating Eq. 6:

$$F_{r,m} = \left[\left(\frac{p_A}{\gamma} + Z_A\right) - \left(\frac{p_B}{\gamma} + Z_B\right) + C_{m,A}\,\frac{V^2_A}{2\,g} - C_{m,B}\,\frac{V^2_B}{2\,g}\right]\gamma\,A \quad \ldots (7)$$

It was shown previously that integrating the momentum equation canceled all the internal forces and left only the external forces. Therefore, $F_{r,m}$ must represent the total shearing stress along the channel boundary between A and B. If this force is replaced by an equivalent negative force on B equal to $\gamma\,h_{f,A-B}\,A$ (in which h_f is an expression in terms of "loss of head due to friction" of the shear at the boundary) the usual form of the Bernoulli equation is derived with the velocity distribution coefficients and the friction term included:

$$\gamma\,h_{f,A-B}\,A = \left[\left(\frac{p_A}{\gamma} + Z_A\right) - \left(\frac{p_B}{\gamma} + Z_B\right) + C_{m,A}\,\frac{V^2_A}{2\,g} - C_{m,B}\,\frac{V^2_B}{2\,g}\right]\gamma\,A \quad \ldots (8a)$$

and

$$\frac{p_A}{\gamma} + Z_A + C_{m,A}\,\frac{V^2_A}{2\,g} = \frac{p_B}{\gamma} + Z_B + C_{m,B}\,\frac{V^2_B}{2\,g} + h_{f,A-B} \quad \ldots\ldots (8b)$$

Instead of replacing $F_{r,m}$ by an equivalent "loss of head force," it can be stated directly in terms of unit shear at the boundary, τ_0. This unit shear is usually expressed in terms of the kinetic energy of the water[5] based on the mean velocity, $\rho\,\frac{V^2}{2}$, by means of a resistance coefficient, λ. The coefficient λ is the same as $\frac{f}{4}$, in which f is the familiar Darcy coefficient of friction.

[5] "The Physics of Solids and Fluids with Recent Developments," Pt. 2, by L. Prandtl, 1936, p. 275.

The total resisting force may then be put equal to the total shear at the boundary:

$$F_{r,m} = \tau_0 P L \quad \ldots\ldots (9a)$$

and since

$$\tau_0 = \frac{\lambda \rho V^2}{2} = \frac{f \rho V^2}{8} \quad \ldots\ldots (9b)$$

substituting values from Eqs. 7 and 9*b* in Eq. 9*a* and transposing,

$$\frac{f \rho V^2}{8} P L = \left[\left(\frac{p_A}{\gamma} + Z_A \right) - \left(\frac{p_B}{\gamma} + Z_B \right) + C_{m,A} \frac{V^2_A}{2g} - C_{m,B} \frac{V^2_B}{2g} \right] \gamma A \quad \ldots (10a)$$

$$\frac{f}{8} \frac{\rho}{\gamma} \frac{P}{A} V^2 = \frac{\left(\frac{p_A}{\gamma} + Z_A \right) - \left(\frac{p_B}{\gamma} + Z_B \right) + C_{m,A} \frac{V^2_A}{2g} - C_{m,B} \frac{V^2_B}{2g}}{L} \quad \ldots (10b)$$

and

$$\frac{f}{8g} \frac{1}{R} V^2 = S \quad \ldots\ldots (10c)$$

By making the proper substitutions for f in Eq. 10*c*, the Chézy and Manning equations are obtained. Substituting $f = \frac{8g}{C^2}$ gives the familiar Chézy equation—

$$V = C \sqrt{R S} \quad \ldots\ldots (11)$$

or substituting $f = \frac{n^2}{0.00858 R^{1/3}}$ gives the Manning equation—

$$V = \frac{0.262 \sqrt{g}}{n} R^{1/6} \sqrt{R S} = \frac{1.486}{n} R^{2/3} S^{1/2} \quad \ldots\ldots (12)$$

Although the substitution of $f = \frac{n^2}{0.00858 R^{1/3}}$ is usually regarded as empirical, it does not affect the rationality of the development of the slope term, which is the concern of this paper. Likewise it is to be emphasized that f is a coefficient of boundary resistance alone and therefore both C and n are coefficients of boundary resistance alone, and are usually considered as such.

The momentum theorem is thus seen to provide a rational method of deriving the Bernoulli, Chézy, and Manning equations in a form which is usable and the one in which they are usually considered. The proper velocity-distribution coefficient to use in such equations is, therefore, $C_m = \frac{\overline{u^2}}{V^2}$.

This can also be shown in another and perhaps more direct manner. It was noted previously that the change in momentum per second can be expressed as a force. This force can be measured directly with a pitot tube, and is equal to $\gamma \frac{u^2}{2g} dA$. Therefore, the total force is

$$F_{r,m} = \frac{\gamma}{2g} \int_0^A u^2 \, dA \quad \ldots\ldots (13a)$$

For this value can be substituted

$$F_{r,m} = \gamma\, C_m \frac{V^2}{2\,g} A \qquad (13b)$$

in which C_m is the coefficient previously defined. Thus, if one were to integrate over a cross section with a pitot tube, the ratio of the average height of water in the tube, above the free water surface, to the height of water in the tube corresponding to the mean velocity would give the coefficient C_m.

Development of the Energy Equation

In contrast to the momentum theorem, the energy principle requires that the work done by the forces be evaluated. Work requires motion. The proper method of applying the energy principle to flowing water is to take a mass of water and determine what takes place as work is done upon it. The work done in moving the mass a given distance could be computed if the mass moved as a solid body. Since the present problem concerns nonuniform velocity, such an assumption cannot be made. The work done during a given time interval can be considered, however. Taking the same mass of water as was considered before (see Fig. 1) and letting the forces act for one second will produce the results shown in Fig. 2. Not only has the mass moved with respect to the

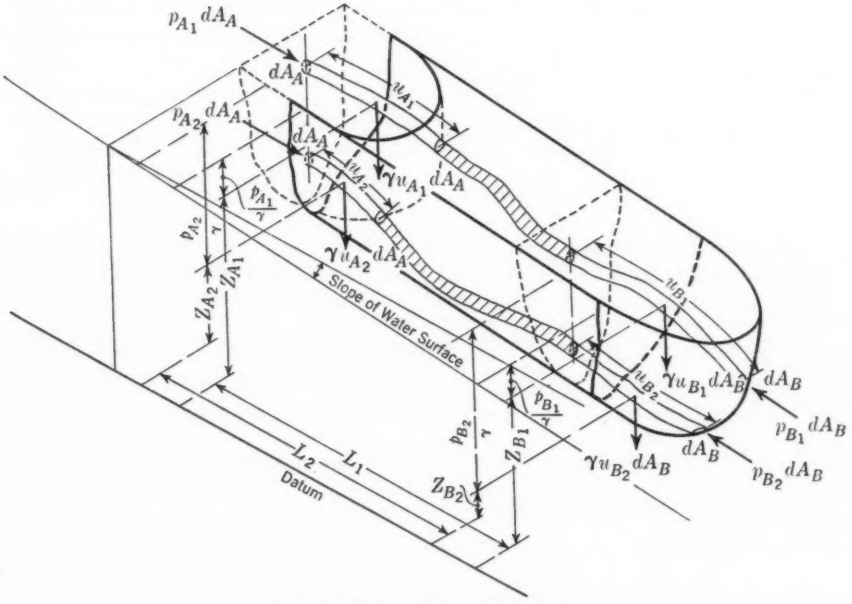


Fig. 2.—Energy Diagram at $t = 1$

channel, but each stream tube has also moved with respect to every other stream tube having a different velocity. Thus, the internal forces acting along the boundaries of the stream tubes do not cancel by action and reaction as in the momentum equation, since there must be residual unbalanced forces that

produce the relative motion between the stream tubes. The integration of the differential equation will sum up these unbalanced internal forces as well as the external forces and the final equation must contain some expression including them.

Proceeding now to evaluate the terms in Eq. 3*b*, the work done by the pressure forces is $p_A\,dA_A\,u_A - p_B\,dA_B\,u_B$. In this case it is not necessary to specify the $dA_A = dA_B$ but the assumption of gradually varied flow must still be applied. Since the volume $dA_A\,u_A$ must equal $dA_B\,u_B$ and may for convenience be expressed as B (bulk or volume) it follows that the volume in between (shaded volume in Fig. 2), although containing different particles of water at $t = 1$ than it did at $t = 0$, has the same energy content throughout the period considered. Thus the work of the gravity forces can be considered as that required to move B from the position at $dA_A\,u_A$ to the position $dA_B\,u_B$, which, as far as the gravity forces are concerned, is $\gamma\,B$ times the vertical component of the distance between them $Z_A - Z_B$. The resisting force will be expressed as $dF_{r,e}$ and the distance through which it moves is $\bar{u}_{A-B}$, the average velocity in the stream tube. Similarly as with the gravity forces, the change in kinetic energy can be expressed as the change in energy of a mass $\rho\,B$ being moved from a position at $dA_A\,u_A$ to a position at $dA_B\,u_B$. Inserting these quantities in Eq. 3*b*:

$$p_A\,dA_A\,u_A - p_B\,dA_B\,u_B + \gamma\,B\,(Z_A - Z_B) - dF_{r,e}\,\bar{u}_{A-B} = \tfrac{1}{2}\,\rho\,B\,(u^2{}_B - u^2{}_A) \qquad (14a)$$

rearranging, and remembering that $B = dA_A\,u_A = dA_B\,u_B$:

$$dF_{r,e}\,\bar{u}_{A-B} = \left[\left(\frac{p_A}{\gamma} + Z_A\right) dA_A\,u_A - \left(\frac{p_B}{\gamma} + Z_B\right) dA_B\,u_B + \frac{u^2{}_A}{2\,g}\,dA_A\,u_A - \frac{u^2{}_B}{2\,g}\,dA_B\,u_B\right]\gamma \qquad (14b)$$

To integrate over the cross section to obtain the total resisting force $F_{r,e}$, it is necessary to make substitutions. The term C_e is defined as the coefficient of energy distribution due to nonuniform distribution of velocity evaluated as $C_e = \dfrac{\overline{u^3}}{V^3}$ or $C_e = \dfrac{\int u^3\,dA}{V^3\,A}$. Still other substitutions must be made in the case of the energy equation, however. During the time period from $t = 0$ to $t = 1$, the original plane cross sections at the ends of the reach have been transformed into warped surfaces, and $\left(\dfrac{p}{\gamma} + Z\right)$ is no longer a constant. As u is likewise a variable, the following substitution is made:

$$C_A = \frac{\int\left(\dfrac{p_A}{\gamma} + Z_A\right) u_A\,dA_A}{\left(\dfrac{p_A}{\gamma} + Z_A\right)_{\text{avg}} V_A\,A_A} \qquad (15a)$$

$$C_B = \frac{\int\left(\dfrac{p_B}{\gamma} + Z_B\right) u_B\,dA_B}{\left(\dfrac{p_B}{\gamma} + Z_B\right)_{\text{avg}} V_B\,A_B} \qquad (15b)$$

By integration, the total resisting force is then:

$$F_{r,e}\,V = \left[C_A \left(\frac{p_A}{\gamma} + Z_A \right)_{avg} V_A\,A_A - C_B \left(\frac{p_B}{\gamma} + Z_B \right)_{avg} V_B\,A_B + C_{e,A} \frac{V^2_A}{2g} A_A\,V_A - C_{e,B} \frac{V^2_B}{2g} A_B\,V_B \right] \gamma \quad \ldots\ldots\ldots\ldots (16a)$$

Dividing by $Q = A_A\,V_A = A_B\,V_B$,

$$F_{r,e} = \left[C_A \left(\frac{p_A}{\gamma} + Z_A \right)_{avg} - C_B \left(\frac{p_B}{\gamma} + Z_B \right)_{avg} + C_{e,A} \frac{V^2_A}{2g} - C_{e,B} \frac{V^2_B}{2g} \right] \gamma\,A \quad \ldots\ldots\ldots\ldots\ldots (16b)$$

This can be arranged into the form of the Bernoulli equation:

$$C_A \left(\frac{p_A}{\gamma} + Z_A \right)_{avg} + C_{e,A} \frac{V^2_A}{2g} = C_B \left(\frac{p_B}{\gamma} + Z_B \right)_{avg} + C_{e,B} \frac{V^2_B}{2g} + \frac{F_{r,e}}{\gamma\,A} \quad \ldots\ldots\ldots\ldots (16c)$$

It is easily seen that a formula such as Eq. 16*c* is not easily adapted to practical solutions of open-channel problems since $F_{r,e}$ involves internal as well as boundary resistance. It could be argued that the Chézy and Manning equations are purely empirical and therefore the slope could be evaluated by means of Eq. 16*b*, substituted in them, and the proper coefficients computed empirically. Such coefficients would necessarily be coefficients of internal resistance as well as boundary resistance and presumably could vary as compared with coefficients of boundary resistance alone. It is doubtful if such a method would yield practical results.

Comparison of Coefficients

Eq. 16*b* is identical in form with the momentum equation (Eq. 7). It differs in that the coefficients C_e are quite different from the coefficients C_m, and the additional coefficients C_A and C_B are also present. It must also be realized that, although $F_{r,m}$ in the momentum equation is the resistance at the boundary only, $F_{r,e}$ in the energy equation includes that force as well as the resisting forces exerted by one stream tube upon another due to their relative motion. It is concluded, therefore, that Eq. 16*b* is not suited to practical application.

Most textbooks on hydraulics which use the energy principle to demonstrate the Bernoulli equation assume a uniform distribution of velocity. With this assumption, in rearranging Eq. 14*a* one could use $B = dA_A\,V_A = dA_B\,V_B$; and the distance through which the resisting force moves is V. Likewise, $V^2_A = u^2_A$ and $V^2_B = u^2_B$ can be substituted. The quantity $\left(\frac{p}{\gamma} + Z \right)$ is now constant over the cross section also, and Eq. 14*a* transforms to

$$dF_{r,u}\,V = \left[\left(\frac{p_A}{\gamma} + Z_A \right) V_A\,dA_A - \left(\frac{p_B}{\gamma} + Z_B \right) V_B\,dA_B + \frac{V^2_A}{2g} V_A\,dA_A - \frac{V^2_B}{2g} V_B\,dA_B \right] \gamma \quad \ldots\ldots\ldots\ldots (17a)$$

which can be integrated without further substitution:

$$F_{r,u} = \left[\left(\frac{p_A}{\gamma} + Z_A\right) - \left(\frac{p_B}{\gamma} + Z_B\right) + \frac{V^2{}_A}{2g} - \frac{V^2{}_B}{2g}\right] \gamma A \ldots . (17b)$$

Eq. 17*b* is similar to Eq. 7 and would be identical if Eq. 7 were based on uniform velocity distribution. In such a case C_m would reduce to 1.00. Since uniform distribution of velocity is assumed, the water in all stream tubes moves forward together, there is no relative internal motion, and therefore $F_{r,u}$ would be the force at the boundary alone, as in the case of the momentum equation.

Summary

Having two identical equations for the flow of water with uniform velocity distribution, one based on the momentum theorem and the other based on the energy principle, it is easy to understand how differences can arise in working backward to nonuniform velocity distribution. Apparently many engineers think of the Bernoulli equation as it is normally written as an energy equation exclusively because it can be obtained using energy principles and assuming uniform velocity distribution. This leads to the desire to correct for nonuniform distribution by introducing the coefficient C_e. However, the Bernoulli equation is also a momentum equation and when so used the proper coefficient for nonuniform velocity distribution is C_m. If it is desired to use the coefficient C_e it must be realized that the other coefficients C_A and C_B must be added and that the nature of the resistance F_r changes also.

The historical background concerning the confusion of coefficients has been presented by B. A. Bakhmeteff,[6] M. Am. Soc. C. E., and the strict hydrodynamical approach was given by G. H. Keulegan[7] at the suggestion of the writer. The present paper is an attempt to interpret the two references cited in terms of demonstrations found in ordinary textbooks.

[6] "Coriolis and the Energy Principle in Hydraulics," by B. A. Bakhmeteff, *Theodore von Karman Anniversary Volume*, Contributions to Applied Mechanics and Related Subjects, 1941, p. 59.

[7] "Equation of Motion for the Steady Mean Flow of Water in Open Channels," by Garbis H. Keulegan, Research Paper RP1488, *Journal of Research*, National Bureau of Standards, Vol. 29, July, 1942, pp. 97–111.

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PAPERS

MILITARY AIRFIELDS

A SYMPOSIUM

PAGE

Construction and Design Problems.
BY JAMES H. STRATTON, M. AM. SOC. C. E. 28

Design of Drainage Facilities.
BY GAIL A. HATHAWAY, M. AM. SOC. C. E. 55

Bibliography.
BY THE SYMPOSIUM AUTHORS 88

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CONSTRUCTION AND DESIGN PROBLEMS

BY JAMES H. STRATTON,[1] M. AM. SOC. C. E.

SYNOPSIS

The procedures in use by the Corps of Engineers, U. S. Army, in the design and construction of airfield pavements, are described in this paper. The design procedures for heavy airplane wheel loads having been in use only since the beginning of the present war construction program, it is too early to forecast the ultimate service behavior of airfields of recent construction. However, to the extent permitted by the pressure of the war construction program and the demands of air traffic, the design and construction procedures have been verified by accelerated traffic tests utilizing heavy, rubber-tired earth-moving equipment. The service performance, under severe operating conditions, of pavements designed and constructed in accordance with these procedures, on the whole, has been extremely gratifying.

Criteria for airfield site selection and for grading and clearance of runways, taxiways, and aprons; investigation and test procedures; and airfield drainage are also discussed.

LOADING CRITERIA FOR MILITARY AIRFIELDS

Military airplane wheel loads adopted by the Corps of Engineers for pavement design are shown in Table 1. Class IV military airfields are exceptional; a wheel loading of 15,000 lb may be considered the minimum military design standard. Loadings greater than 60,000 lb may be anticipated in the near future, but no design provision is being made yet for such loadings.

TABLE 1.—MILITARY AIRPLANE WHEEL LOADS FOR PAVEMENT DESIGN

Class	Wheel load (lb)	Contact pressures (lb per sq in.)	Contact area (sq in.)
I	60,000	75	800
II	37,000	65	570
III	15,000	55	273
IV	5,000	45	111

Impact.—That the buoyancy or wing lift of moving planes markedly reduces rolling impact stresses at runway joints and cracks is indicated by the excellent service performance of runways as compared to apron and taxiways of the same design. The lesser effect of wing buoyancy in reducing such stresses on apron and taxiway concrete pavements results from the slower rate of plane travel on these pavements.

In the design of aprons and taxiways, special allowance is made for vibration stresses caused by the warming of airplane engines, and this allowance is considered as providing for rolling impact on these pavements.

Investigations by the Army Air Forces and the Corps of Engineers disclose that landing impact for good two-wheel landings is not severe, whereas one-wheel landings result in high impact stresses. The generally excellent service

[1] Col., Corps of Engrs., U. S. Army, Arlington, Va.

performance of the parts of runways on which most landings occur indicates that landing impact does not contribute to pavement failures. Hence, stresses due to landing impact are ignored.

Frequency of Loading.—In general, the loading frequency of airfield runway pavements is considerably less than on highways, due both to inherent traffic limitations of airfields (air saturation) and to the natural traffic distribution that results from the relatively greater widths of runways (150 ft to 300 ft).

Stresses Due to Warming of Airplane Engines.—Airplane static loadings and the loadings of airplanes when engines are being warmed induce higher stresses than those produced by rapidly moving planes. This has been verified by noting the relatively large increases in deflections of flexible pavements imposed by standing airplanes and by standing planes, and slowly moving planes, when their motors are being warmed at high speed (dynamic loading). The dynamic loading is taken into account in designing turnarounds, aprons, hardstandings, and taxiways that are subject to the stresses thus caused.

The dynamic loading is adjusted by increasing the loadings, in Table 1, 25% for the design of turnarounds at the ends of runways, aprons, hardstandings, and taxiways subject to normal frequency of use. This load increase assumes a 25% increase in the wheel load distributed over a tire print area (see Table 1) which is also increased by 25%. Warming engines on runways is exceptional, and dynamic loading is not considered in runway design.

Design of Certain Facilities for Limited Use.—In the interest of economy, a reduction in design requirements is feasible for certain airfield pavements, such as stub-end and peripheral taxiways and the pavements of auxiliary and all-over fields which are used infrequently ("limited operation"). For flexible pavements the combined thickness of base course and the pavement required for the specified load assumption is decreased 20%. For rigid pavements a higher working stress is used. Pavements are not designed for both dynamic loading (warming of airplane engines) and limited operation.

Criteria Governing Selection of Pavement Types.—Service aprons and hardstandings are subject to the high shearing stresses that occur when a plane is turned with locked wheels, and to the action of oil and gasoline dripping and spilling. Since bituminous pavements are unsatisfactory for such facilities, as well as for turnarounds at the ends of runways (which are subject to similar high shearing stresses) concrete generally is used. Concrete is not warranted for hardstandings at temporary airfields. Since reasonable maintenance is practicable and economical, for the most part they are constructed of surface-treated base courses designed for the specified loadings. Ordinarily, stub-end and peripheral taxiways are constructed with surface-treated base courses.

Turfing of Airfields.—Where climatic and soil conditions permit, runway and taxiway shoulders are turfed to control dust and erosion. Such turfed runway and taxiway shoulders will support reasonably high loadings under reasonably favorable ground conditions, and thus will reduce the likelihood of accidents when the shoulders are used inadvertently by planes. Where turfing is impracticable or uneconomical, the shoulders are treated with a dust palliative of a texture and color which distinguishes them from the taxiways and runways, and thus prevents their being mistaken for landing and traffic areas.

Concrete Paving

The design of concrete pavements generally is based on the methods of H. M. Westergaard, M. Am. Soc. C. E. (1)(2)(4)[2] which contemplate loads applied at the corners, edges, and the interior of pavement slabs. The Westergaard analysis omits consideration of temperature variations and other influences that tend to change the slab volume, effect of edge thickening, effect of local hard or soft spots in the subgrade, horizontal components of the reactions of the subgrade, and inertia of pavements and of subgrades. However, these factors have a direct bearing on pavement performance, and to the degree of their importance are considered in the design and construction of airfield rigid pavements.

The design of airfield concrete pavements is an empirical application of the Westergaard formulas for interior slab loadings for large tire imprint. A design factor is selected to provide a pavement sufficiently thick to withstand the design stresses at interior corners and under the conditions of load repetition normal at military airfields. Typical design curves for the specified allowable working stresses and wheel loads are shown in Fig. 1. After the modulus of

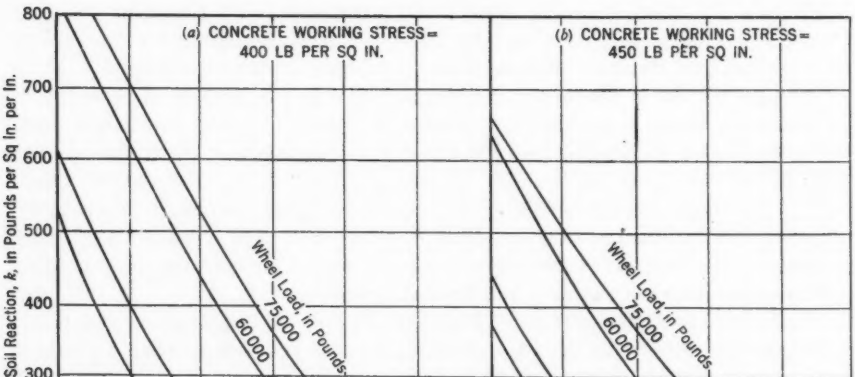


Fig. 1.—Design Curves for Concrete Pavements

soil reaction k has been determined, the required thickness of slab, except at free edges and along longitudinal expansion joints, is taken directly from the design curves. The results of accelerated traffic tests using rubber-tired earth-moving equipment with wheel loads and tire imprint areas comparable to those specified for the design of Class I, II, and III airfields, and of load tests on

[2] Numerals in parentheses, thus: (1), refer to corresponding items in the Bibliography at the end of the Symposium.

concrete slabs of various thicknesses placed on subgrades of various types, were used in selecting the design factors and in checking the design method. Airfield pavement service performance has verified the design method, which is being checked further by additional traffic and plate bearing investigations and by observation of the service behavior of rigid airfield pavements.

Temperature Stresses.—Temperature differences between the upper and lower surfaces of concrete slabs cause warping; the effect of warping varies with the magnitude of the temperature differential. Midday temperature differentials of 24° F in a 6-in. slab, and of 31° F in a 9-in. slab, have been recorded in the vicinity of Washington, D. C. (5). It is probable that greater differentials are experienced elsewhere, as, for example, in the southwestern United States. Temperature differentials and their effects are discussed by Dean Westergaard (6), L. W. Teller, Sr., M. Am. Soc. C. E., and E. C. Sutherland (5), E. F. Kelley (7), and R. D. Bradbury, Assoc. M. Am. Soc. C. E. (8). Mr. Bradbury concludes that, for the purpose of design, a maximum temperature differential of 3° F per inch of slab thickness may be assumed when the top of the slab is warmed by the sun and the bottom is cooled by the subgrade. He suggests a differential of 1° F per inch of slab thickness when, at night, a reverse temperature differential may prevail. The design differentials proposed by Mr. Bradbury are less than those recorded by the Public Roads Administration (5), and probably are considerably less than those that may be expected at some localities.

Actual warping stresses noted by the Public Roads Administration (5), in summer in the vicinity of Washington, D. C., were 320 lb per sq in. for a 6-in. uniform slab, and 416 lb per sq in. for a slab with thickened edge (9 in.-6 in.-9 in.). The investigations of the Public Roads Administration and the analyses by Dean Westergaard indicate that temperature warping stresses may be reduced materially by shortening the slabs. For example, for a 10-ft slab the interior longitudinal stresses are 66%, and the free edge stresses 89%, less than the corresponding stresses in a 20-ft slab.

Excessive warping stresses are avoided by imposing design limitations on slab lengths and by permitting relatively thin slabs for the design loadings when base courses are used to improve subgrade support. This is better practice than to place thick slabs directly on subgrades of low bearing capacity. The relatively low volume of traffic at airfields results in relatively infrequent occurrence of the critical combined stress condition at midday when warping stresses are at a maximum.

Factors Affecting Subgrade Support.—Variations of subgrade reaction as they affect stresses in a slab are regarded by Dean Westergaard (1) and others as not particularly significant. This conclusion is based on values of the subgrade moduli ranging between $k = 50$ and $k = 200$ lb per sq in. per in. The bearing value of a subgrade is influenced by a number of factors, including variations in the soil moisture resulting from fluctuating ground water, seepage through joints, and capillarity—which induces condensation at the underside of the paving slab—and also subgrade deformation due to loading and to temperature warping. Warping of a concrete pavement tends to deform the subgrade, which in turn affects the magnitude and distribution of the stresses in the pave-

ment. Compaction of subgrades and the use of compacted base courses to improve and to develop uniformity of the subgrade support mitigate these adverse influences.

Highway Procedures for Determining the Subgrade Modulus.—In the design of rigid highway pavements it is customary to assume a value for the subgrade modulus generally ranging from $k = 50$ to 200, with $k = 100$ most commonly assumed as representing fair average subgrade conditions. No method of evaluating the subgrade support for concrete pavement design has been devised and generally adopted.

In corner-load testing of concrete slabs on a synthetic subgrade of moist clay, M. G. Spangler, M. Am. Soc. C. E. (9), computed an apparent initial subgrade modulus of $k = 275$ corresponding to a deflection of 0.0232 in. maximum, and an apparent final value of $k = 40$ corresponding to a deflection of 0.0620 in. maximum at 400 to 500 load repetitions. This decrease was the result of deformation of the subgrade under the repetition of load. Although this reduction resulted in a computed increase in slab stress of about 20% to 25%, Mr. Spangler, concurring with Dean Westergaard, concluded that variations of subgrade reaction of this magnitude have relatively little effect upon the stresses in the slab. However, he suggests the need for further investigation. In a discussion of Mr. Spangler's paper, the late Clifford Older, M. Am. Soc. C. E., observes that warping of slabs under field conditions will result in permanent subgrade depression, and that this indicates the fallacy of using stress formulas that are based on the assumption that the subgrade reaction is proportional to slab deflection. Mr. Older concludes that test slabs should be traffic tested to confirm the design assumptions and relationships of load, subgrade reaction, and slab stress.

The stress increases accompanying the decrease of subgrade reaction as noted in the laboratory by Mr. Spangler probably are less than those often observed under field conditions. The result of repeated deflections of a slab, particularly at joints where water has ready access, may be a remolding of a plastic subgrade soil, or a pumping action with a consequent marked loss of support at the locations where slab stresses are most critical. Base courses contribute to the alleviation of conditions destructive to the supporting capacity of a subgrade.

Determination of Subgrade Reaction.—Designs of airfield concrete pavements are based on the modulus of soil reaction k determined by field plate bearing tests. Tests to determine k are conducted prior to design, whenever feasible. The pressure of the war construction program has precluded field bearing observations and other required investigations in some instances and, to estimate the k values, it has been necessary to use, interpretatively, the results of field plate bearing tests at other sites. To represent the subgrade in cut, bearing test areas are stripped of vegetation, scarified, and compacted at optimum moisture to a depth of 6 in. To represent graded fill, test embankments at least 30 in. high are constructed with the same compaction control that is used in the airfield construction. Where low subgrade support is apparent, base courses are placed at selected thicknesses on the compacted test area (both cut and fill) and then evaluated by plate bearing tests. These tests of both sub-

grade and base course are repeated sufficiently often to give representative values. The results of the plate bearing tests are plotted as stress-deformation curves, and the subgrade modulus k_u (and base-course modulus) at optimum moisture is selected as the slope of a chord passing through the origin and the curve at 0.05-in. deflection. This value of deflection and the size of the bearing plates (30 in. in diameter) were selected after a series of bearing and traffic load tests. The selected deformation tolerance of 0.05 in. for the determination of this modulus may not be a proper one for all types of subgrades nor for base courses placed on subgrades to improve the pavement support.

The optimum moisture condition at which the subgrade and base courses are placed does not represent the moisture condition after the passage of time. To evaluate the subgrade support that might obtain as a result of moisture accumulation, principally in the subgrade, field plate bearing tests were made on subgrades that were saturated after initial testing at optimum moisture. The results of these investigations were not satisfactory, and an alternate procedure has been adopted tentatively to adjust for possible decrease in subgrade support due to moisture accumulation. This consists of utilizing the results of laboratory consolidation tests on subgrade samples compacted at optimum moisture, and on duplicate samples similarly compacted which are saturated by immersion prior to testing to represent probable future moisture accumulation and swelling of the subgrade. The duplicate sample is saturated under a restraining load representing the weight of the overlying pavement. The adjusted value of subgrade modulus is determined by the formula:

$$k = \frac{p_s}{p} k_u \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which p is the load intensity on the 30-in. plate that produces a settlement of 0.05 in.; p_s is the load in pounds per square inch required to consolidate the saturated specimen to the same extent as the load of intensity p when applied to an unsaturated laboratory specimen. The adjustment for possible subgrade saturation takes into account neither the attendant remolding of the subgrade, the effects of cyclical swelling and shrinkage of the subgrade, permanent subgrade deformation resulting from slab warping or load application, nor the effects of frost heaving or thawing.

The interpretation of plate bearing test results requires the exercise of judgment based on experience with accelerated traffic tests and upon observation of pavement service performance for extended periods. Such interpretation requires full information on the subgrades, base courses, and concrete which implies extensive laboratory tests of the subgrade, base-course materials, and the concrete; on such factors as the effects of subgrade restraint and of changes in the subgrade induced by change in moisture content; the amount and effect of temperature warping and of frost action and thawing; and the effect of slab dimensions and of stress transfer devices. Field plate bearing tests are so expensive and time consuming, and require the mobilization of such large amounts of equipment for test embankment construction, compaction, load jacking, and base-course construction that such testing often is precluded.

Base Courses for Concrete Pavements.—When used in pavement designs for Class I and II airfields, wheel loading values of the subgrade moduli commonly assumed in highway practice result in excessively thick pavements (10 in. to 14 in.). Such thick slabs obviously are undesirable because of the high-temperature stresses that would result. Furthermore, experience has shown that concrete slabs more than 8 in. thick are inferior in the lower portion, due to difficulties in vibrating the concrete.

The excessively thick pavements for Class I and II airfields required for subgrades of low bearing value are avoided by the use of base courses to improve the support for the pavements. High-quality base materials are less subject to the adverse influences which readily deteriorate the susceptible low-quality subgrades. They also serve to insulate and protect the underlying subgrades, and thus to preserve, to a large degree, whatever value and uniformity of support the subgrades offer. Recent plate bearing investigations have disclosed that improvement of the pavement support by use of base courses is not always evidenced in the values of k as determined by the plate bearing tests. Accelerated traffic tests and pavement service performance, however, demonstrate the value of base courses under concrete pavements.

Allowable Working Stresses for Concrete Pavements.—For normal operation, the allowable working stress of the concrete is determined by dividing the 28-day flexural strength by a design factor of 1.75. This value was chosen after giving due consideration to warping stresses, effect of loading at joints, concrete fatigue, and increased strength of concrete with age. For limited operation a design factor of 1.40 is used. The results of traffic tests and observations of rigid pavement service performance were considered in selecting these design factors.

Airfield Concrete Pavement Sections Used by Corps of Engineers.—Standards of the Corps of Engineers for concrete pavement sections are shown in Fig. 2. Edges are thickened only at free sides and at longitudinal expansion joints. Where slabs have thickened edges (joints), the shaping (troughing) for the thickened edges results both in material disturbance to the subgrade or base course and in reduction in base-course thickness, with resulting reduction in the value of the slab-supporting medium. Since pavement slab stresses are critical at edges and corners, it is imperative that the integrity of the slab support at these locations be especially protected. The shaping of base courses is particularly difficult, and it is probable that the benefits to be expected from the use of base courses are not realized when the base material is disturbed to thicken the edges. For this reason, edges are not thickened at interior joints (except at free longitudinal expansion joints), and adequate stress transfer devices are provided.

Stress Transfer Across Joints.—Observations of paving construction have disclosed the difficulties in providing high quality concrete at construction keys. Honeycombing due to lack of consolidation by spading and vibration of the concrete vitiates the functioning of the keys as devices for stress transfer. Messrs. Teller and Sutherland (5) have shown the need for supplemental transfer devices at keyed joints. Their conclusions and the observed poor workman-

ship at keys have resulted in the decreased use of keys in military airfield pavements.

The widths of pavement strips range generally from 20 ft to 30 ft, the present trend being toward wider lanes. Where strips exceed 18 ft in width, continuous longitudinal dummy joints are run down the center. Tie bars are placed at longitudinal dummy joints in the paving strips adjacent to longitudinal ex-

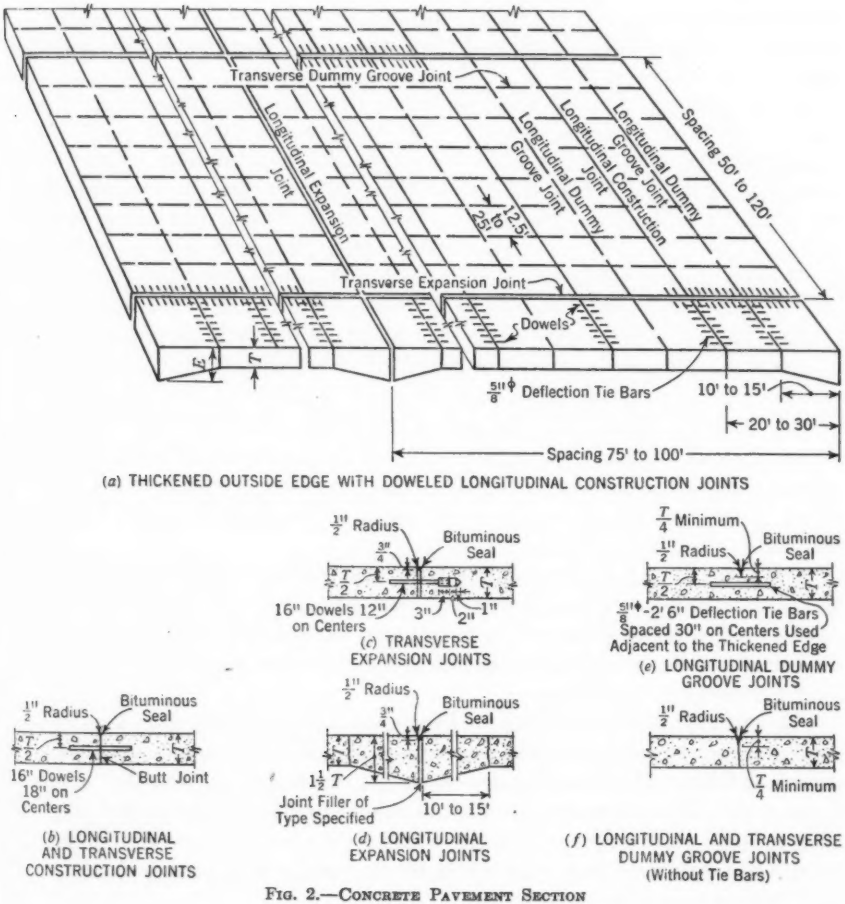


FIG. 2.—CONCRETE PAVEMENT SECTION

pansion joints and outside pavement edges. The provision of stress transfer devices at other joints than those shown in Fig. 2 is left for local consideration.

Joint Spacing.—The spacing of expansion and contraction joints is largely a matter for local determination. The character of the subgrade and base course (including their restraining influences), the climate (particularly daily temperature range), and the season of construction, as bearing on the question of initial shrinkage, are factors that affect the spacing of expansion joints.

Longitudinal expansion joints are laid out generally at intervals of 75 ft to 100 ft, and for widths of 100 ft or less no longitudinal expansion joints are required. Transverse expansion joints are at intervals of 50 ft to 120 ft depending on the judgment of the district engineer. Transverse expansion joints are equipped with dowels that have been painted, greased, and capped at one end.

Transverse contraction (dummy groove) joints are at intervals of 10 to 25 ft, most pavements having spacings of less than 20 ft. The likelihood of high warping stresses due to temperature differentials has an important bearing on the spacing of transverse joints, and the practice is to use short slabs where the differentials may be high.

Construction of Concrete Pavements.—A well-prepared rigid pavement design may be vitiated by carelessness or improper construction, and emphasis is placed on the quality of subgrade and base-course construction; protection of the prepared subgrade, base courses, and the concrete during the construction; the selection of proper equipment for construction; construction sequences; and adequate curing of the concrete. For thick slabs particularly, there is need for improvement in placement methods. Alternate lane construction of multi-strip airfield pavements is conducive to serious disturbance to the subgrade by the operation of construction equipment and by ponding of rain water on the intervening subgrade between completed strips. On the other hand, in successive lane construction serious cracking has resulted from the operation of paving machines mounted at one end on previously placed concrete which has not yet gained sufficient strength to support the paving-machine load. The latter procedure is preferred, however, and the construction schedule is established, where feasible, to permit proper aging of the concrete before imposing the paving-machine loading.

Concrete curing is highly important and construction instructions emphasize the need for protecting the concrete to avoid the adverse effects of temperature warping before strength gain is sufficient to resist warping stresses.

Because of the war shortage, steel reinforcement has been omitted from the rigid pavements of airfields.

Summary of Factors Affecting the Design of Concrete Pavements.—Load, temperature warping, and subgrade reaction are the principal factors affecting the stress condition of concrete pavement slabs. By improving the slab support through the use of base courses it is possible to reduce the theoretical stresses from airplane wheel loadings to values which are commonly accepted in highway practice, and to keep slab thicknesses to dimensions that obviate excessively high-temperature warping stresses.

Subgrades deteriorate and lose their supporting value as a result of moisture accumulation and remolding of the subgrade soil or because of deformation due to loading and to temperature warping. This degradation of the subgrade results in cracking of the overlying pavement. Base courses which improve the support for rigid pavements insulate the underlying subgrades against the adverse influences to which many are highly susceptible. Properly designed base courses are highly resistant to these same influences.

Flexible Pavements

The wide variety of aggregate types suitable for bituminous pavements often permits economies through the use of local materials. The so-called "low type of flexible pavements" is generally unsuitable for military airfields, except for traffic facilities used infrequently. Central hot-plant bituminous concrete construction is preferred for pavements subject to normal use.

Flexible pavement construction for airfields (10) generally requires base courses to distribute stresses to the subgrade. The various formulas in vogue for determining the combined thickness of base course and pavement required for a specified loading generally depend on a rating of the subgrade bearing capacity. The subgrade is usually rated by plate bearing tests.

Bearing plate investigations and accelerated traffic tests using heavy, rubber-tired earth-moving vehicles indicate that flexible pavements on airfields, designed on the basis of subgrades rated at deflections commonly considered acceptable, are not satisfactory for present-day airplane loadings. It has been found infeasible to fix a maximum value for the deflection tolerance that would be applicable in rating subgrades of various soil types for flexible pavement design. Moreover, the effects of moisture change, rate of load application, stress repetition, and frost heaving and thawing cannot be evaluated directly by the plate bearing method of rating subgrades. The effects can be evaluated, if at all, only by empirical correlation of the results of a large number of plate bearing tests with the service performance of pavements on a wide variety of subgrades. These facts led to the consideration of other design methods.

California State Highway Department Method of Design.—The standard method of design adopted by the California State Highway Department (13) has been accepted for the design of flexible pavements at airfields. To extend the experience gained in California to the loadings required for airfield pavements, the design curves for wheel loadings in excess of 12,000 lb, shown in Fig. 3, were prepared by the engineering staff of the Chief of Engineers based upon its design, construction, and operating experience and with the advice and assistance of O. J. Porter and Arthur Casagrande, Members, Am. Soc. C. E. A series of accelerated traffic tests on pavements with varying thicknesses of base courses, founded on several distinctive types of subgrades to represent the range of subgrade soil types which might be encountered, resulted in substantial confirmation of the design curves tentatively established for wheel loadings in excess of 12,000 lb.

As the result of more recent accelerated traffic tests to determine the relationship of the values of the California bearing ratio (CBR) to required combined thicknesses of pavements and base courses, the curves have been revised in the range covering cohesionless subgrades to reduce the required combined thickness of pavements and base courses.

Evaluation of Subgrade.—The CBR of a subgrade soil is its relative stability as determined by an arbitrary penetration test and is expressed as a percentage of the stability value of a selected crushed stone. The penetration values for the crushed stone have been established. The CBR of a subgrade is tested by

forcing a circular piston, 3 sq in. in area, into the subgrade soil in a cylindrical test mold at the density and moisture conditions to be expected after the passage of time as a result of moisture accumulation, and noting its load deformation characteristics. The procedure has been fully described by Mr. Porter (11). It is applicable to highway construction methods used in the State of California and to conditions and soils in that state.

Inconsistent test results on certain of the many types of soils encountered in the construction throughout the continental United States, and elsewhere, have made it necessary to modify the California test procedure to insure representative results consistent with the character of the construction. As more tests are performed, it may be necessary to alter further the CBR test procedure.

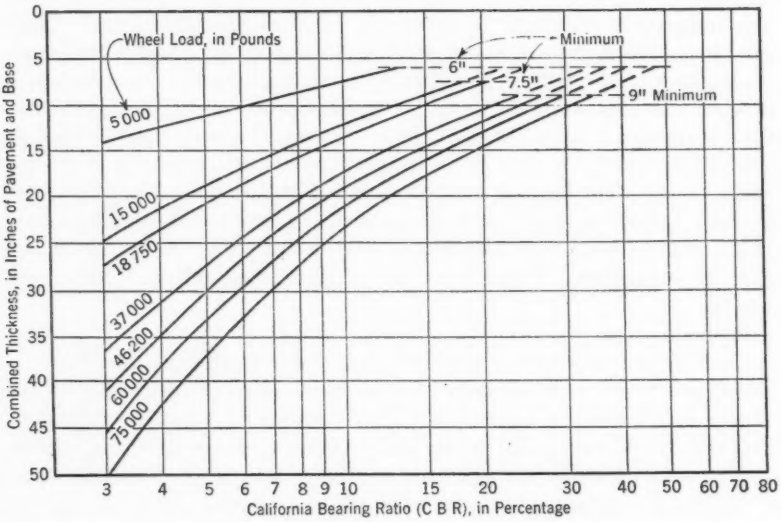


FIG. 3.—CURVES FOR THE TENTATIVE DESIGN OF FLEXIBLE PAVEMENTS (CALIFORNIA METHOD)

Instead of compacting soils in the test mold under a load of 2,000 lb per sq in. (termed static load), as in the California method, it has been found advisable to tamp the soil in the test mold as is done in laboratory testing for density and moisture control for field compaction. The control test for most soils is a modification of the well-known Proctor test. The densities of some soils when compacted by the static load vary as much as 10% from the densities obtained by tamping the soil in the mold. Static loading compaction of some soils results in nonuniform densities or a crust-like surface condition in the upper zone. This is the part of the specimen which is penetrated by the test piston, and the effects influence the CBR test results.

Adoption of a tamper similar to the Proctor tamper, to compact the soil in the CBR test mold, has led to other modifications in the testing procedure and also in the testing equipment. Since the CBR test results are greatly affected

by the surface condition of the specimen penetrated, the test mold has been redesigned to facilitate inverting the mold and penetrating the bottom end of the sample, which is relatively smooth, free of crusts, and of relatively uniform density.

For some soils the California method of soaking the CBR test specimen to represent the density and moisture conditions that may obtain in the subgrade after the passage of time results in a very soft, mushy testing surface. This condition is prejudicial to the results of the CBR test and is not representative of the actual subgrade moisture and density conditions. Soils of this type now are soaked by permitting the water to rise from the bottom until expansion ceases, or until free water appears on the surface.

Tests of granular materials in accordance with the original procedure give CBR values which result in excessive base thickness requirements. The weight of the annular ring that encircles the piston when it penetrates the soil sample, and which is used to represent the confining effect of the overlying base and pavement, has been increased to represent the restraining effects of the overlying base and pavement material.

The value of the CBR method of design lies in the correlation with pavement performance by the California Highway Department over a period of thirteen years for the range of loadings common to highway practice, and with the results of accelerated traffic tests conducted by the Corps of Engineers with loadings comparable to those of present-day airplanes (see Table 1).

Incomplete laboratory tests in process suggest the following tentative conclusions:

(*a*) The moisture content of low to medium plastic soils at the time of compaction in the test mold affects the CBR. The CBR decreases with increase of moisture content.

(*b*) A change in density affects the CBR value for most cohesionless and low to medium plastic soils.

(*c*) The magnitude of the penetration surcharge (annular ring) has a marked effect upon the CBR of cohesionless soils, and some effect on low plastic soils.

(*d*) The CBR is increased for some soils if the length of the sample is less than 4 in.

(*e*) Erratic CBR values result from testing soils containing a high percentage of particles greater than $\frac{1}{4}$ in. in size. It is considered necessary to make numerous tests of such soils.

Design for Capacity and Limited Operation.—The required combined thickness of pavement and base course, based on the CBR test of the subgrade soil, is taken from Fig. 3 unless an additional thickness of base course is needed to insulate the subgrade against frost action. The CBR method of design is based on the performance of flexible highway pavements under a wide variety of conditions over a period of years. The combined thickness of base course and pavement requirement, as specified by the design curves for a given loading, provides for the depreciation in the subgrade bearing value by subgrade saturation, subgrade moisture change, stress repetition, and other influences

(except frost action) which tend to decrease the stability of the subgrade material. The total combined thickness given by Fig. 3 is used for capacity operation, but for pavements subject to limited operation it is reduced by 20%.

Where field compaction of the subgrade is impracticable or inadvisable, or where unusually weak strata underlie the compacted subgrade layer, the CBR tests are conducted on undisturbed subgrade samples. The effects of saturation on the uncompacted subgrade are determined by duplicate tests on saturated, undisturbed specimens of the subgrade.

Design of Base-Course Layers.—Suitable base courses can be constructed using soils of relatively low CBR in the deeper layers. This practice is often economical if materials with high CBR are costly to process or must be transported from more distant sources. The required combined thickness of base material and pavement above any given base-course layer depends upon the CBR of the base-course layer and may be determined from Fig. 3. If the base course is designed for several layers of material, the economic thickness of each layer is determined. If a material of high bearing value is used for the full depth of base a lesser thickness is required than if lower CBR material is used in the lower portions of the base. If this opinion is verified by further tests, this fact will permit a change in the CBR design curves.

The base course directly under the pavement should be sufficiently stable to withstand the high stresses produced in the zone directly under the wheel of a plane. The required stability depends upon the type and thickness of pavement, the action and effect of a moving or skidding wheel, the type of plane, etc., and cannot be determined by Fig. 3. For highway pavements used by heavy trucks, the base-course material immediately under a bituminous pavement should have a CBR of at least 80%. Preliminary tests disclose that materials of lower CBR directly under airfield pavements would probably give satisfactory performance for airfield traffic conditions. When material with a CBR of 80% is not available locally, and transportation facilities are inadequate for importing suitable material, the requirements for the top 6 in. of base course have been tentatively established as follows:

Gross weight of plane, in pounds	Minimum allowable CBR for upper base material
10,000 to 30,000	50%
30,000 to 74,000	65%
74,000 to 120,000	80%

Recent investigations refute the results of the preliminary tests and indicate that material of less than 80% CBR should not be used directly under pavements when the subgrade soil is plastic.

Pavement Thickness.—The cross section of a bituminous pavement may comprise several courses, depending on the total thickness required, economic use of materials, practical construction limitations, the possibility of stage development of the airfield, etc. Pavement thicknesses are designed with a surface course, or surface and binder courses, to resist displacement and shear under the design wheel loads, giving consideration to the type of bituminous material used. Pavement thicknesses are not less than 1.5 in., for wheel loads up to 15,000 lb, and not less than 3 in., for wheel loads up to 37,000 lb. For

wheel loads exceeding 37,000 lb the thicknesses are designed in accordance with the requirements in each specific case, but in general do not exceed 6 in.

Pavement Quality.—A high density in surface courses is required for resistance to water absorption and raveling under traffic, and is achieved by the proper gradation of aggregates, thorough mixing with the bitumen, and thorough compaction to produce a low void ratio. Bituminous surfaces are compacted from 90% to 95% of the theoretical maximum density, depending on the type. Grades of bitumen are selected which will remain soft and pliable and will promote healing of incipient cracks induced by weathering and wear, without benefit of a substantial amount of traffic kneading.

The non-skid requirements for runway surfaces are less important than for highway surfaces, because of the primary difference in type of traffic, alinement, and grades. Pavements with rough surface textures provided by the coarse graded mixtures, which have been widely favored for highways because of their high non-skid properties, are not needed for runways, and are undesirable from the standpoint of excessive tire wear. The reasonably fine-grained, gritty texture produced by coarse sand or small stone chips, gravel, or crushed slag, used as cover aggregate for seal coats, is desirable for runways.

Subgrade and Borrow Investigations

Explorations are made to determine the location of ledge rock, elevation and slope of the water table, and soil profiles, and to provide representative soil samples for testing. The subgrade soil survey is extended to a depth equal to at least twice the equivalent diameter of the tire contact area, but in no case less than the maximum depth of frost penetration or less than 4 ft below the final elevation of grading at each point where grading is required.

Laboratory tests on subgrade soil samples include mechanical analysis and soils classification, Atterberg limits, moisture content, natural density, CBR, expansion, permeability, and compaction control. Shear strength and consolidation tests are made if the pavement is to be supported on embankment fill. Soils are classified in accordance with the Casagrande Soil Classification (12). Local classification often is used to supplement this general classification, where it is necessary to describe the soil adequately.

Compaction Tests and Control

Compaction of soils is controlled, wherever applicable, by the AASHO Method (American Association of State Highway Officials) modified as follows to give densities comparable to those obtained by specified field compaction equipment: The weight of the rammer or metal tamper is 10 lb instead of 5.5 lb; the tamper is dropped from a height of 18 in. above the sample instead of 12 in.; and the samples are compacted in five equal (approximately 1-in.) layers instead of three layers. The densities of some cohesionless sands and gravels as obtained by the Modified AASHO Method often are not representative of obtainable field densities, and the control specimens of such soils usually are compacted by tamping, static pressure, or vibration alone or in combination.

Fill is constructed in 9-in. layers, loose measurement, and compacted to at least 90% of the maximum unit weight at optimum moisture, except that di-

rectly beneath the base course or pavement where a base course is not provided, the subgrade is compacted to at least 95% of the maximum unit weight at optimum moisture, for a depth of not less than 6 in. In cut areas most subgrade soils are scarified and recompacted so the unit weight of the top 6 in. is at least 95% of the maximum unit weight at optimum moisture. Scarifying, reworking, and compacting certain types of clay soils often result in lowering the bearing value below that of the natural clay in place. For this type of material, tests are made to determine the relative bearing values of the recompacted soil and of the undisturbed soil; and, if the undisturbed soil has the higher bearing value, construction operations are conducted to produce the least possible disturbance of the subgrade.

Treatment of Soils with Expansive Characteristics

Natural subgrades with expansive characteristics are compacted at a moisture content, not necessarily the optimum moisture, to a unit weight which will minimize expansion. The selected moisture content and unit weight are approximately equal to those of an expanded sample after the soil has been compacted at optimum moisture and saturated.

Pavement Design for Frost Action

The stability of some soils is greatly reduced by frost action, the loss of stability and extent of heave depending upon the type of soil and the drainage conditions. Several investigators have established limiting grain-size curves defining the boundary between frost-action and nonfrost-action soils for uniform grading. The most generally accepted method of insuring pavement or base-course stability over a potential frost-action soil is to provide sufficient thickness of insulating material. Where the required thickness of base-course material as determined by CBR is not sufficient to preclude frost penetration in the subgrade, additional thickness of insulating material often is required. In general, to insure stability of flexible pavements and surface-treated bases, the combined thickness of pavement and base material not subject to frost action is made equal to the average depth of frost penetration as determined from local records.

Base Courses

Wide varieties of gravels, base-course materials, sands, and gravelly and sandy soils as well as crushed rock, shell, caliche, etc., are used alone or blended to provide base courses. Processing by crushing and screening often is required to produce the required grading.

Where crushed gravel or stone is used for base courses directly under concrete pavements, a reasonably smooth top surface of base course is provided to reduce restraint to the pavement. Base-course materials sometimes are stabilized with commercial admixtures such as portland cement, cut-back asphalts, emulsified asphalts, and tars. Such admixtures are most advantageous with relatively sandy or friable soils.

All base courses are compacted to the maximum density obtainable with heavy field compaction equipment. If the Modified AASHO Method is ap-

plicable, the base course is compacted to at least 95% of the maximum density at optimum moisture content.

Special Investigation of Airfield Pavements

Various tests and investigations have been made by the Corps of Engineers for the purpose of improving design and construction methods. Procedures for the investigations and tests were devised to determine conclusions which might be applied to the current construction program. The investigations fall into three categories: Laboratory tests, field bearing tests, and accelerated traffic tests using rubber-tired earth-hauling equipment with wheel weights comparable to the airplane wheel loadings given in Table 1.

Laboratory Investigations.—The special laboratory investigations comprised all the routine tests ordinarily made for site selection and pavement design. The extensive use of the CBR test soon disclosed that the results are influenced by sample density; variation in density of the sample; moisture condition of sample; method of saturating sample to represent moisture conditions that may result in the subgrade after the passage of time; weight of the annular surcharge used to represent the confining effects of the overlying pavement and base course; and rate of penetration. Although considerable laboratory investigational work has been done to adjust the CBR testing procedure to permit proper evaluation of these variables, the results have not been fully correlated with the service performance of flexible pavements.

Field Bearing Test.—A large number of plate bearing tests were made to develop the design method for flexible pavements and to develop a satisfactory procedure for determining the modulus of subgrade reaction which could be used in the design of concrete pavements.

The investigation of design methods for flexible pavements included plate bearing tests on subgrades, pavements, special test sections, and on actual highways in use for several years. It was concluded that the supporting power of subgrades could not be evaluated by plate bearing tests which, for their interpretation, predicate a universal value of allowable deformation for all types of soil. Various values of allowable deformation, ranging from 0.1 in. to 0.5 in., have been proposed by different authorities for rating the unit bearing capacities of subgrades by plate bearing tests.

A study was made to determine if subgrades could be rated by determining in each test the point at which the rate of deformation is accelerated under load application. This would correspond to the yield point on the stress-deformation curve. Fig. 4 is a typical stress-deformation curve obtained by a plate bearing test. The yield point, as can be seen by the curve, is not definite, but can be assumed to be at the point indicated. For this particular test, a bearing value between 15.5 lb per sq in. and 40 lb per sq in. will result if the usual assumptions as to allowable deformation are made.

These studies indicate that the bearing value probably could be selected at some value for the deflection (deformation) less than that at the yield point. However, various factors, including moisture condition and density of the subgrade soil at the time of testing, and the effects of moisture change, stress repetition, and other influences on the behavior of each subgrade under in-

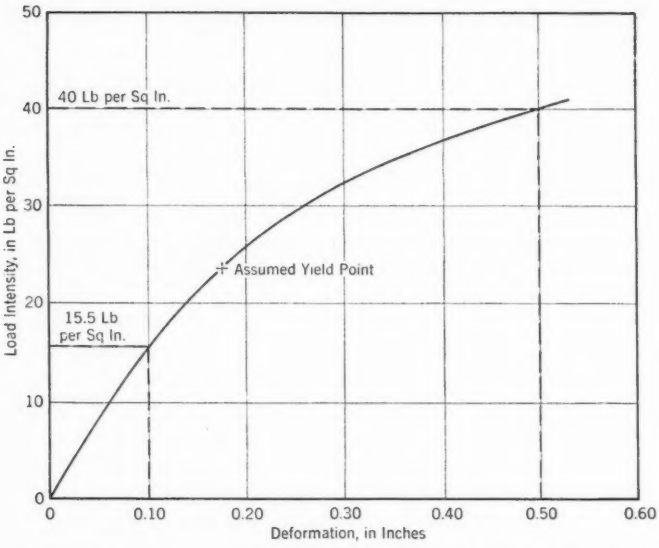


FIG. 4.—LOAD-DEFORMATION CURVE OF A PLATE 30 IN. IN DIAMETER PLACED ON THE SUBGRADE (LOADS APPLIED IN INCREMENTS OF 5 LB PER SQ IN. AND ALLOWED TO ACT UNTIL DEFORMATION CEASED)

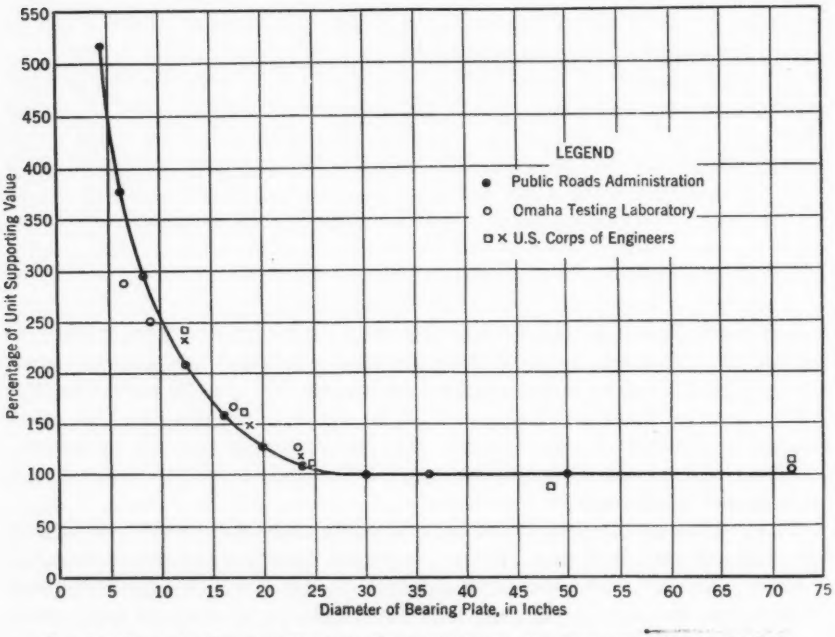


FIG. 5.—RELATION BETWEEN BEARING PLATE DIAMETER AND SUPPORTING VALUE OF SOIL

vestigation, present complications in the selection of a value for the allowable deformation which, in each instance, would represent an evaluation of these influences on the supporting capacity of the subgrade. Small promise of a satisfactory solution was offered by a general investigation of the plate bearing method of rating subgrades with the view of establishing a satisfactory design procedure. Moreover, the pressure of the war construction program made such an investigation infeasible.

Plate bearing tests were made to determine the relation of the area of the plate to the bearing value of the subgrade (see Fig. 5). The results agree closely with those obtained by other investigators and show that, for all practical purposes, the bearing value is the same for all plates with a diameter greater than 30 in. To eliminate the effect of area, a circular plate 30 in. in diameter was selected to determine the modulus of soil reaction for the design of concrete pavement.

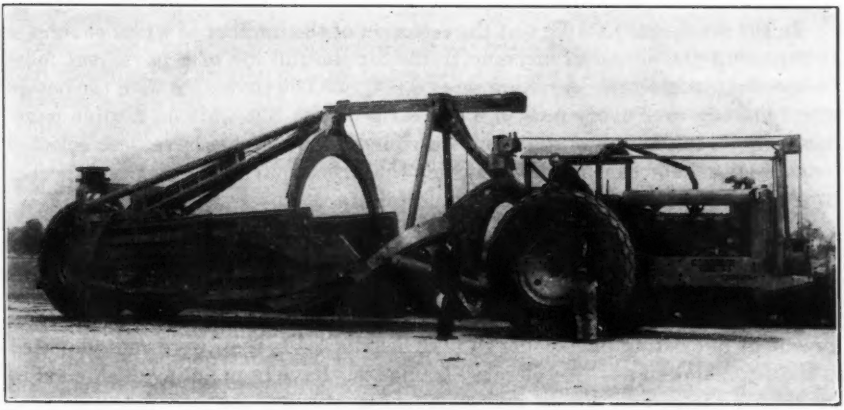


FIG. 6.—EQUIPMENT FOR ACCELERATED TRAFFIC TESTS WITH WHEEL LOADS OF 37,000 LB TO 60,000 LB

Accelerated Traffic Tests.—Fig. 6 is a picture of the rubber-tired earth-hauling equipment which is being used to produce traffic wheel loads from 37,000 lb to 60,000 lb. Smaller equipment similar to this is used to produce loads of 15,000 lb to 30,000 lb. Fortunately, the tire imprint of such equipment is practically equal to that of airplane tires carrying similar loads. This type of equipment is used in making accelerated traffic tests of specially constructed test sections of both flexible and concrete pavements and also of actual airport pavements.

The severity of the accelerated traffic test with respect to fatigue stresses in the concrete pavements, and the rapidity of the repetitions of stress in the tests on both rigid and flexible pavements, as compared to the frequency of their occurrence under service conditions, must be weighed in evaluating the test results.

Fig. 7 is a cross section of an embankment constructed for testing CBR design procedures for flexible pavements. This type of construction permits

economies and insures a uniform subgrade condition. On such a test section deflection measurements were made with planes standing, with planes moving at slow rates with engines turning at high rates, and with planes standing with engines revolving at high rates to represent the condition of engine warming. For the results of testing one such flexible pavement embankment, reference is made to the Stockton Test Report (3).

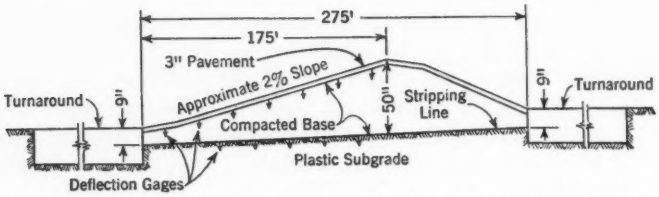


FIG. 7.—LONGITUDINAL SECTION OF EMBARKMENT TO TEST FLEXIBLE PAVEMENT

In the accelerated traffic test the selection of the number of wheel coverages to represent the action of airplane traffic for the full life of a pavement must be based on judgment. A minimum of 3,000 to 5,000 coverages with the design wheel loading over every part of a test strip 5 ft to 8 ft wide on flexible pavements and a minimum of 10 ft wide on concrete pavements have been selected tentatively as representing the use which the airfield pavement may be expected to receive in the course of its normal useful life.

REINFORCEMENT OF EXISTING AIRFIELD PAVEMENTS

The increasing number of heavier planes has created a demand for airfields of greater load capacity. In some instances this demand has required reinforcement of existing pavements to take heavier loads than were contemplated in the original designs. Accelerated traffic tests have been an invaluable aid in evaluating the capabilities of such pavements.

Existing pavements are reinforced and pavements that have failed are rebuilt usually by overlay construction, which involves the use of bituminous pavements, or concrete on concrete or bituminous pavements. Where concrete is laid on concrete, an intermediate slippage course of $\frac{3}{4}$ in. to 2 in. of bituminous stabilized sand is used. The joints and joint spacing of the overlay concrete pavement are made to correspond, as far as practicable, with the joints of the concrete pavement being reinforced.

DRAINAGE OF AIRFIELDS

There has been a gradual development of airfield drainage from the era of the early turfed field relying entirely on the existing natural drainage features, through the period of combination storm and subsurface drains in stone-filled trenches along runway pavements, to the present use of separate storm and subsurface drainage systems with the surface runoff modulated by controlled ponding. These changes in design have been necessitated largely by the exacting requirements of military operations as described by Mr. Hathaway in the companion paper of this Symposium.

Surface Drainage Systems.—The storm drainage system must be adequate in design and areal extent to remove a selected design storm without damage to airfield facilities, undue saturation of the subsoil, or significant interruption of normal traffic. Furthermore, the drainage must be so designed that storms greater than the design storm will not cause undue damage or serious interruption of traffic.

A drainage system plan for a typical airfield is shown in Fig. 8(*a*). For economy in drainage installation there should be a proper relationship between the grading plans and the drainage design to obtain the maximum use of temporary storage of storm runoff.

Depending on the functional requirements of the airfield, storm drainage systems usually are designed to remove runoff from rainfall having a certain average frequency of occurrence. The infiltration capacity estimated to be characteristic of a given soil following a rainfall of one hour serves as the most convenient index to the probable volume of loss through infiltration during the design storm.

Subsurface Drainage Systems.—Subsurface drainage frequently can be eliminated by providing an adequate and properly designed surface drainage system. However, where the ground water is above the frost line or causes soft, boggy conditions in areas where soil stability is required, subsurface drainage is necessary. Two principal types of subsurface drain installations are encountered, the more frequent being a single line of drains adjacent to either side of the runway pavement and so constructed as to drain the pavement base courses and subgrade (see Fig. 8(*b*)). The second type of installation consists of a grid system of drains, not unlike a farm tile drainage system.

The design of a subsurface drainage system is a very approximate procedure based largely on the judgment of the engineer.

Failures in the operation of combined storm and subsurface drainage systems have led to the general requirement that the two systems be so designed that surface runoff cannot backflow into the subsurface drains and result in saturation of pavement base courses and subgrades.

Collectors on pavements are used only to handle the melt of heavy accumulations of snow stacked at the edges of runways. The design of such a collector system is shown in Fig. 8(*c*), the connection being made directly to the storm drains along the edge of the landing strip, either at a manhole or by a special tee in the line. It is imperative that this type of collection system shall not contribute to the moisture accumulation in the subgrade and base course.

Selection and Site Planning of Airfields

Airfield sites are selected by the Engineer Department in close coordination with representatives of the Army Air Forces. The requirements of the Air Forces which influence selection and planning of the airfield embrace: (1) Type of aircraft, including wheel load and other characteristics affecting the design of runways, the size of parking areas, hangars, etc.; and (2) type of unit to be accommodated (bomber, fighter, or other) whose different operating procedures demand different facilities. Upon these items are based the number, direction, and dimensions of runways, taxiways, parking aprons, hardstandings, and other

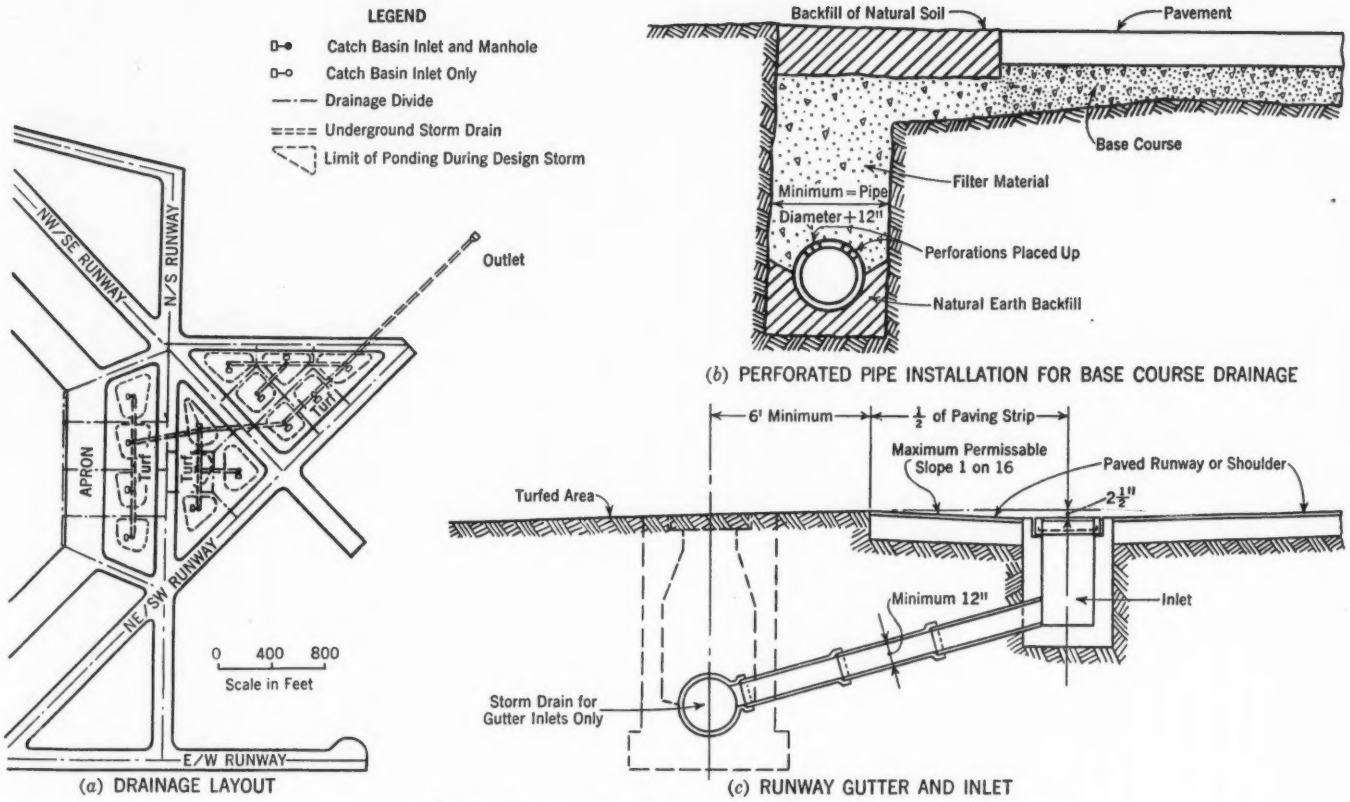


FIG. 8.—PRIMARY STORM DRAIN SYSTEM FOR AN AIRFIELD

facilities. These, in turn, determine the area of the airfield, which normally varies between 1,000 and 1,500 acres for non-dispersed layouts, and proportionally more for dispersed fields. To accommodate constantly increasing demands of new equipment, all new stations are planned with a view to expansion of housing and airplane landing and parking facilities.

Two other main factors influencing site selection and design of airfield layouts are the physical condition of the site and provisions for safety. Crosswind velocities as great as ten miles per hour do not seriously affect the landing or taking off of military aircraft, but winds of greater velocity present serious problems unless the aircraft can be landed or taken off within approximately 22½° of the wind. For this reason, when laying out runways it is necessary to obtain accurate wind information and to interpret it correctly. Although flat terrain is ideal for aircraft operations, it often presents drainage problems that can be solved only by a compromise between the flatness required for aircraft operation and the slopes required for proper drainage. This must be accomplished with a minimum of grading to conserve time, cost, and use of critical equipment. In the choice of a site and in the design of the airfield, the topography of surrounding areas is considered with a view to permitting clear approaches to all runways and future expansion of initial facilities without excessive additional cost of grading or land acquisition.

Safety Provisions.—Safety is provided in: (1) Approach zones, (2) clear zones, (3) lateral clearances from runways, taxiways, aprons, and hardstandings, and (4) building restrictions (see Fig. 9). Approach zones for runways and other prepared landing strips are trapezoidal areas two miles long, 1,500 ft wide at the inner end, 4,000 ft wide at the other end, and symmetrical about the extended axis of the runway or landing strip. Within these limits all obstacles are cleared which would project above a plane sloping upward at the rate of 40 horizontal to 1 vertical, except at certain types of schools and auxiliary fields where a 1 on 30 plane applies. Wherever a clear zone is required, the approach zone begins at its outer limit; otherwise, it begins at the end of the runway or landing strip.

Fundamental regulations for the design of roadways, clear zones, and approach zones are shown diagrammatically in Fig. 10. Except at auxiliary fields and at certain special-purpose airfields, a clear zone usually is required at either end of the runway or landing strip, and extends 1,000 ft in the direction of the center line of the runway or landing strip prolonged.

Lateral clearances from runway center lines, taxiways, aprons, and hardstandings to movable objects (aircraft, vehicles, railway cars, etc.) and to fixed obstacles (buildings, trees, rocks, and terrain irregularities) are prescribed (see Fig. 10) to assure the elimination of all objects that may constitute hazards to the safety of aircraft. Buildings are not permitted between the ends of runways and the boundary of the reservation in the area extending 750 ft each side of the runway center line prolonged, even though such buildings would not project above the approach plane of 1 on 40.

Layout, Dimensions, and Grades of Runways.—Where practicable, runways are laid out to make either 90° or 45° angles with one another, permitting landings and take-offs within 22½° of any prevailing wind. Runways usually are

FIG. 8.—PRIMARY STORM DRAIN SYSTEM FOR AN AIRFIELD

(a) DRAINAGE LAYOUT

oriented with the eight principal directions, but this may be varied to decrease the amount of grading required, to permit clear approach, or to aline runways in directions best suited to future extension.

Current prescribed dimensions for runways call for 7,000-ft length at sea level, of which only 5,000 ft is initially graded and paved, the remaining 1,000 ft at either end constituting the clear zone. The normal width is 500 ft of graded area, of which the center 150-ft strip is paved. Sometimes, 75-ft

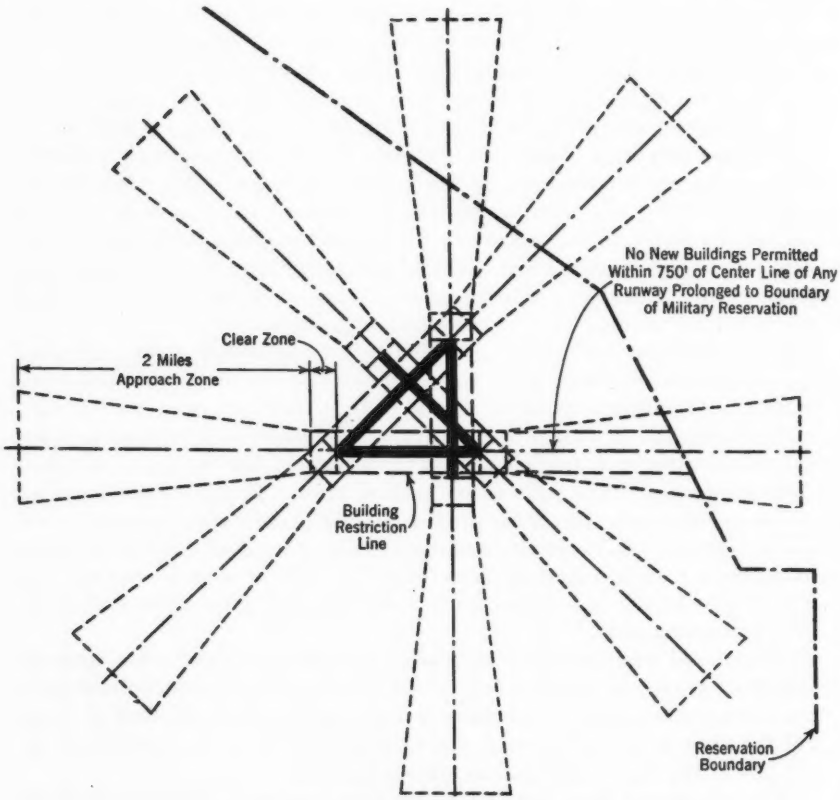
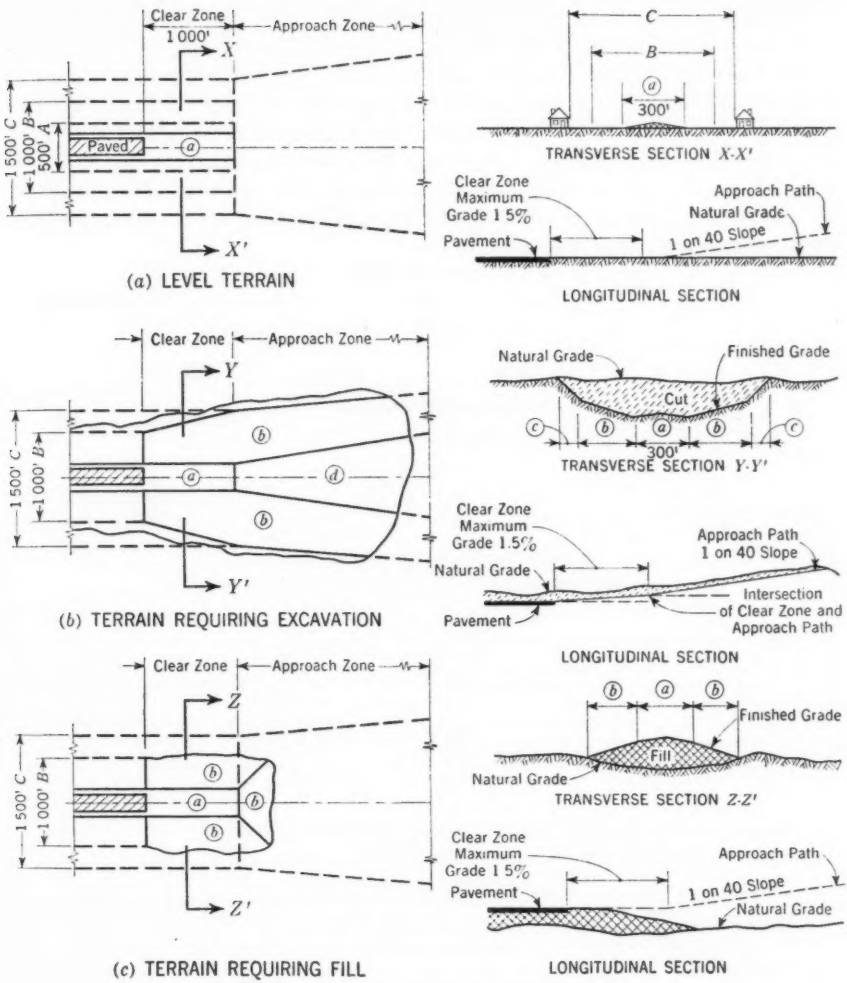


FIG. 9.—SAFETY PROVISIONS IN AIRFIELD DESIGN

shoulders on either side of the paved strip are authorized. For every 1,000 ft above sea level, runway length is increased 500 ft.

Maximum allowable grades are as follows: For runways, 1.5% longitudinal and transverse; for turfed all-over fields, 3% in any direction (except where the field is included within a runway field, when 1.5% in any direction applies); and, for surfaced all-over fields, 1.5%. When runways are designated for 120,000-lb gross load, the maximum allowable longitudinal grade is 1%, although 0.5% is preferred.



SAFETY PROVISIONS

A 500' Strip – Graded Area

B 1 000' Strip, (Ground Loop Area) No Obstacles of Any Kind.

C 1 500' Strip, (Safety Clearance Zone) No Fixed Obstacles (Buildings, Trees, Rocks or Terrain Irregularities) Which Constitute Hazards to Safe Operation of Aircraft.

No Obstacles, Natural or Artificial, High Enough to Protrude Into the Approach Path, Permitted in the Approach Zone

GRADING REQUIREMENTS

(a) Smooth Graded – Compacted and Turfed. Maximum Longitudinal Grade 1.5% Maximum Transverse Grade 1 5%

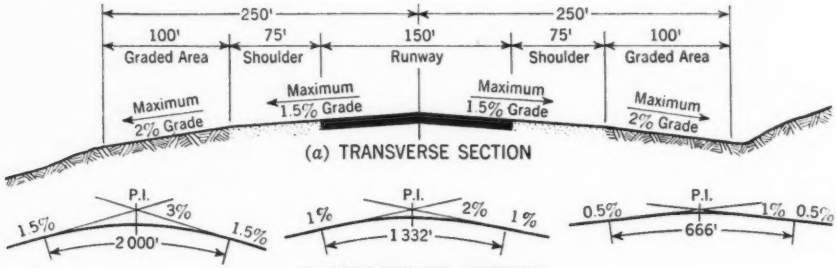
(b) Rough Graded – Maximum Longitudinal Grade 1 5% Maximum Transverse Grade 10%

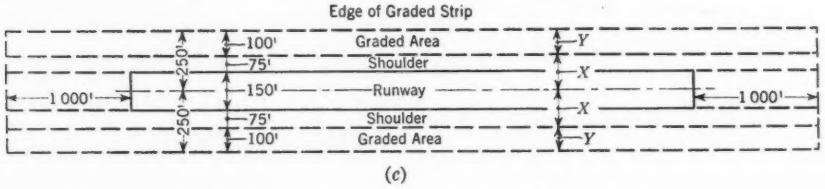
(c) Rough Graded – (Where Necessary) to Provide Slope of Natural Repose

(d) Rough Graded – To Slope of Approach Path. Rough Grade All Excavated Areas in the Approach Zone

FIG. 10.—REGULATIONS FOR THE DESIGN OF RUNWAYS, CLEAR ZONES, AND APPROACH ZONES

Grade change tolerances are illustrated in Fig. 11. The minimum distance between points of intersection (P.I.) at grade changes is 1,000 ft. Parts of a runway having different longitudinal grades are joined by a vertical curve at least 666 ft long for each per cent of the algebraic difference between the two grades. This relationship is maintained for a distance of at least 150 ft from the center line of the runway. Runways are graded for unobstructed visibility between any two points 10 ft above the surface of the runway. If the runway is longer than 7,000 ft, clear visibility is maintained between any two points 10 ft above the surface and 7,000 ft apart.





(c)

Maximum Longitudinal Grade in Area X or Y = 1.5% (1% for 120 000 Lb Wheel Load)
Maximum Transverse Grade in Area Y = 2%; in Area X = 1.5%

Longitudinal Grade Changes in Area X { Minimum Distance Between P.I's. = 1 000'
Minimum Vertical Curve = 666' per Each 1% Algebraic Difference

Longitudinal Grade Changes in Area Y at Edge of Graded Strip { Maximum Algebraic Difference = 2%
Minimum Curve = 300' (Minimum Distance Between P.I's. = 300'

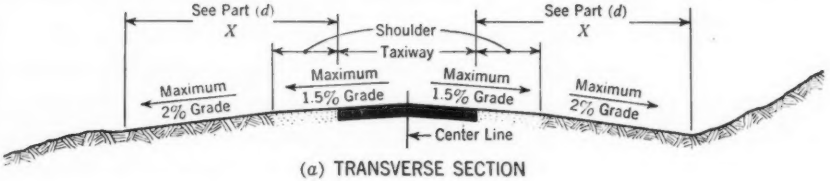
FIG. 11.—PERMISSIBLE GRADE CHANGES FOR RUNWAYS

Airplane Parking Areas.—Although frequently it is feasible to park aircraft on turf, the wear resulting from traffic, propeller blasts, and spillage of oil and gasoline, often renders the turf unusable, and paved parking spaces are necessary. Parking spaces usually are large aprons in the vicinity of maintenance and repair facilities; however, individual hardstandings sometimes are provided in lieu of a center parking apron.

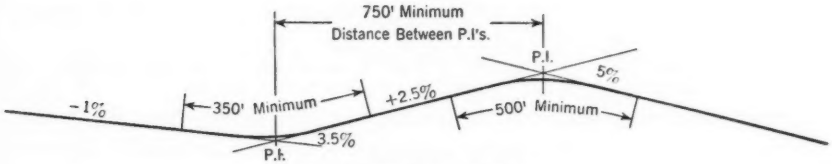
Where conditions permit, the parking apron is placed parallel to the runway upon which the greatest volume of traffic is anticipated, thus shortening the amount of taxiing required by aircraft and permitting full observation of traffic from the control tower. The maximum allowable grade on aprons is 1.5% in any direction. At hardstandings, a small servicing apron is constructed ad-

jacent to the maintenance and repair buildings. Size of parking apron is based upon the number and type of aircraft to be parked on it.

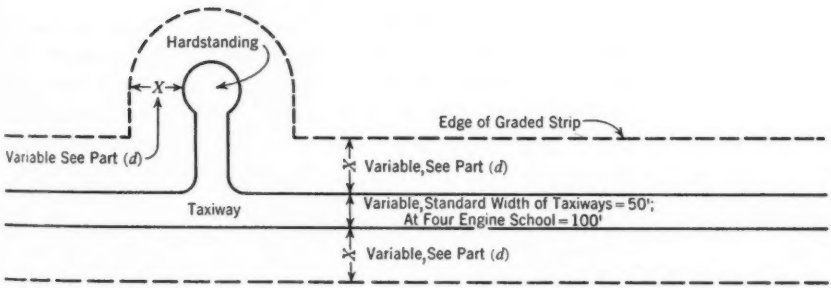
Taxiways.—Ordinarily, taxiways are laid out in straight lines, with angle turns not sharper than 90°, and avoid crossing runways or other taxiways. The standard width for taxiways is 50 ft, with a 12.5-ft shoulder on each side.



(*a*) TRANSVERSE SECTION



(*b*) LONGITUDINAL SECTION
SHOWING TWO EXAMPLES OF PERMISSIBLE GRADE CHANGES



(*c*)

Length of Vertical Curve		Maximum Grades		Gross weight of plane (lb)	*X* (ft)
%[a]	Length[b]	Description	%		
		Algebraic Difference—			
1	300 ft	Planes, 120,000 lb	3.5		
2	300 ft	Planes, 74,000 lb[c]	5.0	30,000	40
3	300 ft	Grades for Taxiways—		74,000	60
4	400 ft	Longitudinal	2.5	120,000	80
5	500 ft	Transverse	1.5		

[a] Algebraic differences between percentage grades. [b] Length of vertical curve required; minimum length, 300 ft. [c] 74,000 lb or less.

(*d*) Standard Dimensions

Fig. 12.—Permissible Grade Changes for Taxiways

Maximum allowable grades are 2.5% longitudinal and 1.5% transverse. Portions of taxiways having different grades are connected by vertical curves having a minimum length of 100 ft for every per cent of algebraic difference between the grades. Vertical curves of less than 300 ft are not permitted under any circumstances, and the minimum distance between points of grade intersections is not less than 750 ft. Examples of permissible grade changes are shown in Fig. 12.

Summary

The trend toward heavier wheel airplane loadings has led to a reexamination of the methods used in the design of airfield pavements. As a result of this reexamination it has become apparent that testing procedures for the evaluation of subgrades and for the economical use of base courses and pavements must be conducted under carefully controlled conditions if efficient and economical designs are to be attained.

Good designs may be vitiated by poor construction and high quality construction control (inspection) is essential.

Accelerated field traffic tests and service performance of airfields have verified the methods of design and construction described. Continuing observations of pavement performance may lead to further revisions and refinements in the design and construction procedures.

Proper design and construction of drainage facilities are essential both to effective pavement performance and the safe use of airfields.

Site layouts must be based on uniform standards with respect to grades and clearances. These standards have been progressively raised as size and landing speeds of airplanes have increased.

DESIGN OF DRAINAGE FACILITIES

BY GAIL A. HATHAWAY,[3] M. AM. SOC. C. E.

SYNOPSIS

The primary purpose of this paper is to suggest procedures for the design of drainage facilities for airfields utilizing surface pondage for temporary storage of runoff from high intensity storms of short duration. Diagrams have been developed by means of which relationships can be determined between drain-inlet capacities, maximum storage volumes, and the duration of rainfall, for the frequency curve selected, that will produce the maximum rate of runoff for the specific area involved. Standard rainfall intensity-duration curves applicable to the continental area of the United States were derived to represent average rates of rainfall for various durations that have the same average frequency of occurrence. The relationship of overland flow and peak runoff rates to various standard rainfall intensity-duration curves is demonstrated and a typical layout and sample computations for the drainage system of an airfield have been included.

GENERAL

A drainage system for an airfield normally serves three functions: Interception and diversion of surface and ground-water flow originating from lands adjacent to the airfield area; removal of surface runoff from the airfield; and removal of ground-water flow (gravitational water) from the airfield area. The respective facilities may be combined or separate depending upon local conditions.

To assure stable soil conditions, subsurface drainage of an airfield is essential in many cases. The need for, feasibility of construction of, and probable efficacy of subsurface drainage should be determined by a study of soil and ground-water conditions at the proposed airfield, but, in general, the efficient removal of surface runoff by a carefully designed storm drainage system will minimize the need of extensive subsurface drainage. Inasmuch as the scope of this paper pertains primarily to surface drainage, discussion of subsurface drainage is limited to the foregoing comments.

The design capacity of the storm drainage systems should be adequate to accomplish the following objectives within the range of economic feasibility, with proper consideration being given to the purpose and importance of the particular airfield:

(*a*) Removal of surface runoff that would result from the selected "design storm," without damage to field facilities, undue saturation of subsoil, or significant interruption of normal traffic;

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(*b*) Removal of surface runoff from storms greater than the "design storm" with the minimum damage to field facilities and with shortest interruption of normal traffic that is practicable;

(*c*) Maximum reliability of operation practicable under all climatic conditions;

(*d*) Minimum maintenance and operation difficulties and expense; and

(*e*) Adaptability to future expansion of airfield drainage facilities with the minimum of expense and interruption of normal traffic.

The importance of careful planning in the preparation of grading plans for an airfield scarcely can be overemphasized. In many instances, the need for expensive underground drains in sections of a field may be avoided by grading the surface to facilitate natural drainage. In other cases required drain capacities may be reduced greatly by shallow ponding basins in intermediate areas between the runways where runoff from intense short-duration rainfall may be stored for brief periods of time without interfering with the use of runways and other essential areas. On the other hand, improper planning of grading details may cause flooding of an important area in one section of the airfield while the drainage facilities in other sections of the field are taxed to only part of their capacity.

The major limitations on grading are as follows: Maximum longitudinal and transverse grade for runways and aprons is 1.5%, except in the case of fields used by four-motored planes, where the maximum longitudinal grade for runways is 1%. A clear graded shoulder should be provided along each side of each runway to a minimum width of 250 ft from the runway center line. The selected transverse grade for the runway pavement is continued to the edge of the stabilized shoulder, 75 ft from the edge of the runway pavement. Beyond this point the shoulder may be sloped longitudinally to drain into inlets located at least 250 ft from the runway center line. In this area the maximum longitudinal and transverse grade of 1.5% is necessary to develop the ponding areas at drain inlets. Along the line of inlets, generally located about 250 ft from the center line of the runway, a maximum allowable grade change not to exceed a 2% algebraic difference is permitted. Grade changes along this line should be connected by vertical curves of not less than 300 ft. Turfed all-over fields which are used only by light-weight aircraft with traffic dispersed over the entire area, have a permissible maximum grade in any direction of 3%. The maximum change in grade which may be made within a distance of 100 ft is 1%.

It is common practice to design storm drains to provide for disposal of runoff from rainfall having certain average recurrence intervals or frequency of occurrence. The rainfall intensity corresponding to a particular frequency of occurrence varies appreciably with the duration of the rainfall, the average rate for a period of 5 min usually being several times as high as the average for a period of 1 hr. A study of rainfall intensity-frequency data for a large number of precipitation stations indicates that there are fairly consistent relationships between the average intensity of rainfall for a period of 1 hr and the average rates of comparable frequency for shorter intervals, regardless of the geo-

One-Hour Rainfall, in Inches, to Be Expected Once in 2 Years
One-Hour Rainfall, in Inches, to Be Expected Once in 5 Years
One-Hour Rainfall, in Inches, to Be Expected Once in 10 Years
One-Hour Rainfall, in Inches, to Be Expected Once in 25 Years
One-Hour Rainfall, in Inches, to Be Expected Once in 50 Years
One-Hour Rainfall, in Inches, to Be Expected Once in 100 Years

FIG. 13.—ONE HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN A GIVEN NUMBER OF YEARS

graphical location of the stations or frequency of 1-hr rainfall. Consequently standard curves may be developed to express the rainfall "intensity-duration" relationships with an accuracy satisfactory for airfield drainage problems.

"Standard rainfall intensity-duration curves" were derived for use in airfield drainage problems in the following manner:

(*a*) The area of the United States was divided into 2° quadrangles;

(*b*) Using data from charts compiled by the late David L. Yarnell, M. Am. Soc. C. E. (15), as shown in Fig. 13, and other available information, 1-hr representative rainfall intensities for each 2° quadrangle were plotted against average rates of comparable frequency for intervals of 5, 10, 15, 30, 120, and 240 min, respectively, using data for average frequency of occurrence of once in 2, 5, 10, 25, 50, and 100 years, respectively.

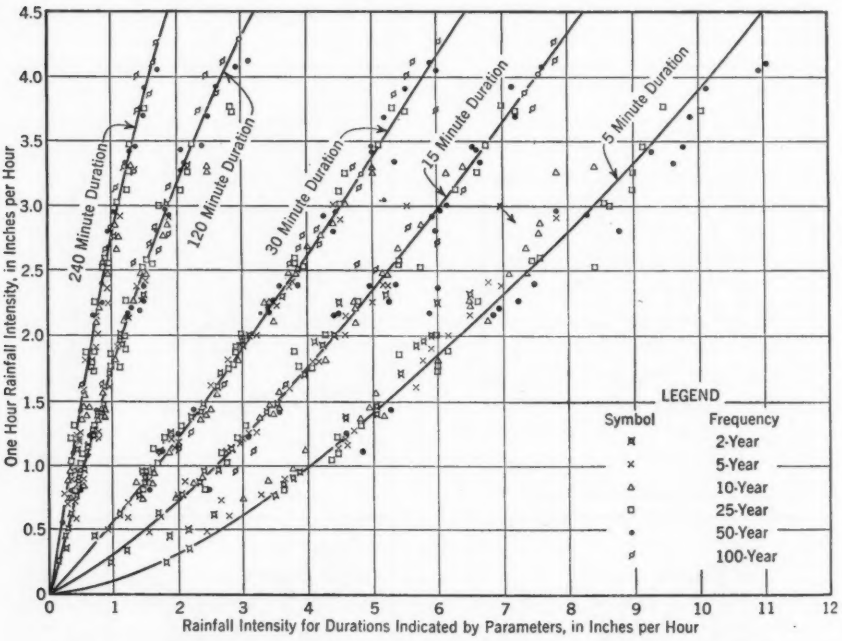


FIG. 14.—RELATION OF 1-HR RAINFALL INTENSITIES TO INTENSITIES FOR DURATIONS OF 5, 15, 30, 120, AND 240 MIN

(*c*) Curves, representing approximately the average of the points plotted in step (*b*), were drawn (see Fig. 14), and data scaled from these curves were replotted in the form shown in Fig. 15. Results of the study indicate that the curves in Fig. 15 may be adopted as general standards in airfield drainage problems with a satisfactory degree of accuracy. For convenience in reference these curves are numbered to correspond with the 1-hr rainfall intensity represented.

Design Storm Criteria

The term "design storm criteria" refers to the rainfall intensity-duration curve adopted as a basis of design for determining the capacity of storm drain facilities for a particular area. The rainfall intensity-duration curves are derived to represent average rates of rainfall for various durations that have the same average frequency of occurrence. Such intensity-duration curves are computed from data for a number of storms; consequently, all of the intensities represented by a particular curve may not be expected in the same storm—at least not with the frequency indicated. For example, if curve No. 2.0 in Fig. 15 represents a 10-yr rainfall frequency at a particular station, it is implied that on the average of once in 10 years a rainfall will occur averaging 3.1 in. per hr for a period of 30 min. This is also shown in Fig. 14 as the point where the 2.0-in. 1-hr rainfall intensity intersects the 30-min duration curve. Although the precipitation continues for a 30-min period at the rate of 3.1 in. per hr, the rain that falls in 30 min totals only 1.55 in. Similarly it is to be expected that on the average of once in 10 years rainfall will average 4.4 in. for a period of 15 min. The precipitation may end at 15 min or continue at some rate equal to or lower than curve No. 2.0, and the intensity for periods less than 15 min may be equal to or less than curve No. 2.0.

The rainfall intensity-duration value required to produce the maximum drain-inlet discharge corresponding to a given design storm criterion, taking into consideration all drainage area characteristics and ponding effects, is referred to herein as the "design storm rainfall." To simplify the preparation of generalized diagrams for estimating drain-inlet capacities required under various conditions for design storm runoff, the following approximations pertaining to the rainfall intensity-duration relations have been accepted:

(*a*) Each standard rainfall intensity-duration curve shown in Fig. 15 is assumed to represent average rates of rainfall for separate storms of various duration that have the same average frequency of occurrence; and

(*b*) Rainfall is assumed to be of uniform intensity during each individual storm, and to end at the duration corresponding to the particular rate on the given intensity-duration curve.

The design-storm frequency serves as an index to the average frequency with which some portions or all of the storm-drain system will be taxed to capacity. The selection of a design-storm frequency for a particular project must be based primarily upon judgment, with consideration of the purpose and importance of the given airfield, and such economic and engineering limitations as may exist. Design-storm frequencies ranging from 2 to 10 years are generally adopted for airfields depending, of course, upon the importance of the field as determined by the class and volume of traffic to be accommodated, the necessity for uninterrupted service and similar factors. It should be observed that, after having selected the design-storm frequency for the particular airfield involved, computations must be made to determine the duration of rainfall for the frequency curve selected that will produce the maximum rate of runoff for the specific area involved. The duration of rainfall required to produce the

maximum rate of runoff will depend primarily upon the length of overland flow, taking into account surface detention, roughness factor, and other surface runoff characteristics.

The design-storm frequency alone is not a reliable criterion of the adequacy of storm drain facilities. In many instances, storms appreciably more severe than the design storm may cause very little damage or inconvenience, whereas in other cases flooding of important areas may result. It is generally advisable

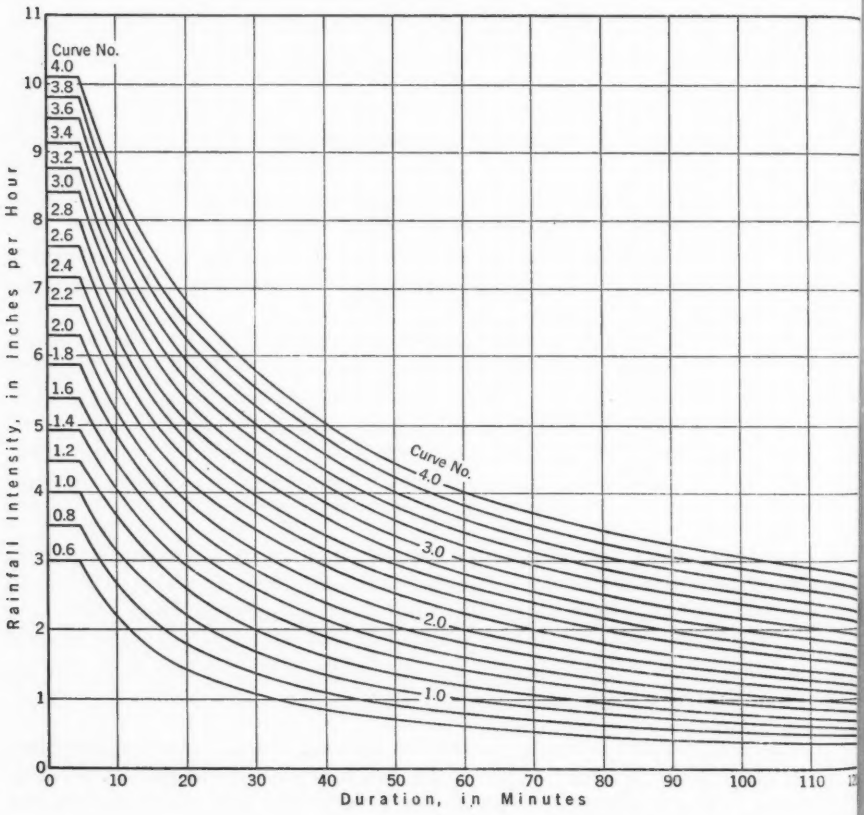


FIG. 15.—STANDARD RAINFALL INTENSI... DUR...

to investigate the probable consequences of storms more severe and less frequent than the design storm, before making a final decision regarding the adequacy of drain-inlet capacities.

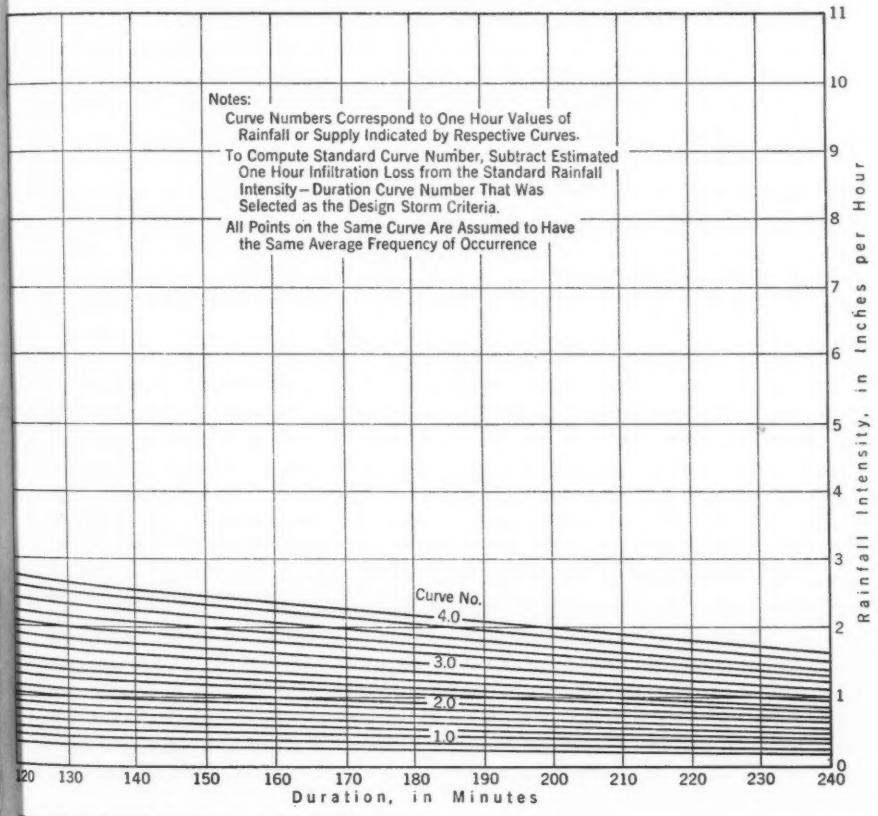
NOTATION

The following letter symbols, used in this paper, conform essentially to American Standard Letter Symbols for Hydraulics[4] prepared by a Committee

[4] ASA—Z10.2—1942.

of the American Standards Association, with Society representation, and approved by the Association in 1942:

I = factor of turbulence;
K = a constant dependent upon characteristics of the drainage area involved;
L = length of an elemental strip of turfed, bare, or paved surface, in a direction parallel to the maximum slope;


Notes:
Curve Numbers Correspond to One Hour Values of Rainfall or Supply Indicated by Respective Curves.
To Compute Standard Curve Number, Subtract Estimated One Hour Infiltration Loss from the Standard Rainfall Intensity–Duration Curve Number That Was Selected as the Design Storm Criteria.
All Points on the Same Curve Are Assumed to Have the Same Average Frequency of Occurrence
Curve No.
4.0
3.0
2.0
1.0
Rainfall Intensity, in Inches per Hour
Duration, in Minutes
120 130 140 150 160 170 180 190 200 210 220 230 240
0 1 2 3 4 5 6 7 8 9 10 11

DURATION CURVES; OR, STANDARD SUPPLY CURVES

M = an exponent dependent upon the type of overland flow;
n = coefficient of roughness;
Q = drain capacity, in cubic feet per second, at a designated point;
Q_d = drain-inlet capacity, in cubic feet per second;
q = rate of overland flow at the lower end of an elemental strip of turfed, bare, or paved surface, in inches per hour or in cubic feet per second per acre of drainage area;
q_d = drain-inlet capacity, or maximum rate of outflow from a ponding basin, in cubic feet per second per acre of tributary drainage area;

q_p = peak runoff rate, in inches per hour or cubic feet per second per acre of drainage area;
S = slope of surface, or the hydraulic gradient;
t = time, or duration, in minutes; time from the beginning of supply:
t_c = critical duration of supply; that is, the duration of rainfall excess (rate of supply) corresponding to a given "standard supply curve" that would produce the maximum rate of outflow from a given drainage area, taking into account surface detention and the surface runoff characteristics;
t_d = time required for water to travel from a specified inlet to a given point in the drainage system;
t_l = critical duration of supply for an elemental strip of effective length L;
t_r = duration of supply, in minutes;
t_σ = duration corresponding to the assumed supply rate σ;
X = ratio of $\frac{q_p}{q_d}$; defined by Figs. 22 and 25;
y = depth, in inches; and
σ = rate of supply, or rainfall in excess of the rate of infiltration.

Infiltration Capacity

In view of the several variations that affect the infiltration capacity of a given soil, it is impracticable to determine accurately the infiltration capacity that should be assumed to apply during storms of various durations and frequencies. The rate of infiltration is a variable that changes not only during the course of a storm but also during a season. Moreover, the infiltration rate changes with the type of soil structure, soil cover, temperature (air, soil, and water), moisture content of soil, turbidity of the water, and the amount of organic matter in the soil. The use of cover on the land, such as grass turf, serves as a protection from the impact of rain, retards the rate of runoff and thereby reduces the velocity of overland flow and turbidity, thus permitting a greater infiltration of water into the soil. Rates of infiltration on bare soil can be expected to be considerably less than for turfed areas. For use in connection with the design of storm drains for a particular airfield, the infiltration capacity that is estimated to be characteristic of the given soil, following a rainfall of 1 hr, serves as the most convenient index to the probable volume of loss through infiltration during the design storm.

The term "rate of supply" (σ) refers to the difference between the rainfall intensity and the infiltration capacity at the same instant of a particular storm. To simplify computation procedures, the rainfall intensity and the infiltration capacity are assumed to be constant during any specific storm. On this premise, the rate of supply during a particular storm would also be uniform. The average rates of supply corresponding to storms of different lengths and the same average frequency of occurrence may be computed by subtracting estimated infiltration capacities from rainfall intensities represented by the proper curve in Fig. 15. For convenience and since no appreciable error results, "standard supply curves" are assumed to be the same shape as the curves in

Fig. 15. The "standard supply curve" number corresponding to a given "standard rainfall curve" number is computed by subtracting the estimated 1-hr infiltration value from the 1-hr rainfall quantity.

Overland Flow Equations

The term "overland flow," as used herein, relates to surface runoff resembling sheet flow before it has reached a defined channel or ponding basin. Robert E. Horton, M. Am. Soc. C. E., has developed the following general equation that indicates the rate of overland flow to be expected from a uniform rate of rainfall-excess, or "rate of supply," assuming flow characteristics ranging from fully turbulent to laminar (18):

$$q = \sigma \tanh^M \frac{M+1}{M} (\sigma K)^{1/M} \frac{t}{60} \quad \ldots\ldots (2)$$

In Eq. 2, q is the rate of overland flow at the lower end of an elemental strip, in cubic feet per second per acre, or inches per hour; σ is the rate of rainfall in excess of infiltration, in inches per hour; M is an exponent dependent upon the type of overland flow; K is a constant dependent upon characteristics of the drainage area involved; and t is the time from the beginning of supply, in minutes. According to Mr. Horton, this equation is strictly rational for 75% turbulent flow ($M = 2.00$) and quasi-rational for other degrees of turbulence from 33% to 100%. The constant K involves slope, roughness and length of overland flow, and percentage of turbulence, and may be expressed by the equation:

$$K = \frac{1{,}020 \sqrt{S}}{I n L} \quad \ldots\ldots (3)$$

In Eq. 3, S is the surface slope; I is the factor of turbulence; n is the coefficient of roughness; and L is the length of an elemental strip of turfed or paved surface in a direction parallel to the maximum slope. The exponent M is equal at 3.00 for laminar flow and $\frac{5}{3}$ for fully turbulent flow; therefore, the factor of turbulence I is given by the equation:

$$I = \frac{3}{4}(3.0 - M) \quad \ldots\ldots (4)$$

For average airfield surfaces, 75% turbulence, corresponding to $M = 2.00$ in overland flow, may be assumed although the percentage of turbulence would be lower for a "dense" grass cover and somewhat higher for very smooth pavement. Substitution of the value $M = 2.00$ in Eq. 2 gives the following formula for the rate of overland flow corresponding to 75% turbulence:

$$q = \sigma \tanh^2 \left[0.922\, t \left(\frac{\sigma}{n L} \right)^{0.50} S^{0.25} \right] \quad \ldots\ldots (5)$$

The factor n in the preceding formulas has the same meaning for overland flow as it has for channel flow but is made up differently. Mr. Horton points out that, in the case of a stream channel, n shows the combined resistance effect

of true rugosity, bends, noneffective cross section in eddies, bays, and deep holes, and expansion and contraction of cross section. All of these factors except true boundary rugosity may be of small importance in a stream channel, but they are often the major factors in case of overland flow. Available data indicate that the following values of the coefficient of roughness n are approximately representative of surfaces encountered in airfield drainage problems:

Surface	Value of n
Smooth pavements	0.02
Bare, packed soil, free of stone	0.10
Poor grass cover, or moderately rough bare surface	0.20
Average grass cover	0.40
Dense grass cover	0.80

In certain approximate computations of overland flow rates, average roughness values for inlet drainage area have been used. The average roughness factor applicable to a drainage area consisting of a combination of paved and unpaved surfaces (or any combination of surfaces having dissimilar roughness characteristics) may be estimated by selecting values of the coefficient n applicable to the respective surfaces and weighting the values in proportion to the surface areas.

The series of overland flow curves for paved and turfed areas shown in Fig. 16 were computed by the use of Eq. 5, assuming S equal to 1% and n equal to 0.02 for pavements and 0.40 for turf. However, these curves may be applied to surfaces having other roughness factors or surface slopes by using, instead of the actual length L of the surface involved, a hypothetical length that is greater or less than the actual length by a sufficient margin to compensate for the difference between the correct values of n and S and those used in preparing the curves shown in Fig. 16. The values of L may be converted easily by the use of Fig. 17, as follows:

(*a*) *Fig. 17*(*a*).—The rate of overland flow indicated by Eq. 5 is a function of the product of $n\,L$; hence, any combination of n and L values that give the same product will result in the same rate of discharge. Thus, a surface having a roughness factor n of 0.40 and a length L of 100 ft would produce the same rate of overland flow as one having an n-value of 0.20 and a length of 200 ft, other factors being the same. The curves in Fig. 17(*a*) represent similar relationships for various values of n.

(*b*) *Fig. 17*(*b*).—The discharge rate represented by Eq. 5 is also a function of the quotient $\frac{S^{0.25}}{L^{0.50}}$. Therefore, the overland flow curves in Fig. 16, computed for a slope of 1%, may be applied to a surface having any other slope, if compensating corrections in the value of L are made in accordance with Fig. 17(*b*).

Inflow Hydrographs

The rates of runoff from turfed areas of various lengths that would result from continuous and uniform rates of rainfall-excess (rates of supply) are represented by the overland flow rates in Fig. 16. For example, with a continuous and uniform rate of supply (σ) of 2 in. per hr and a length of 100 ft, the

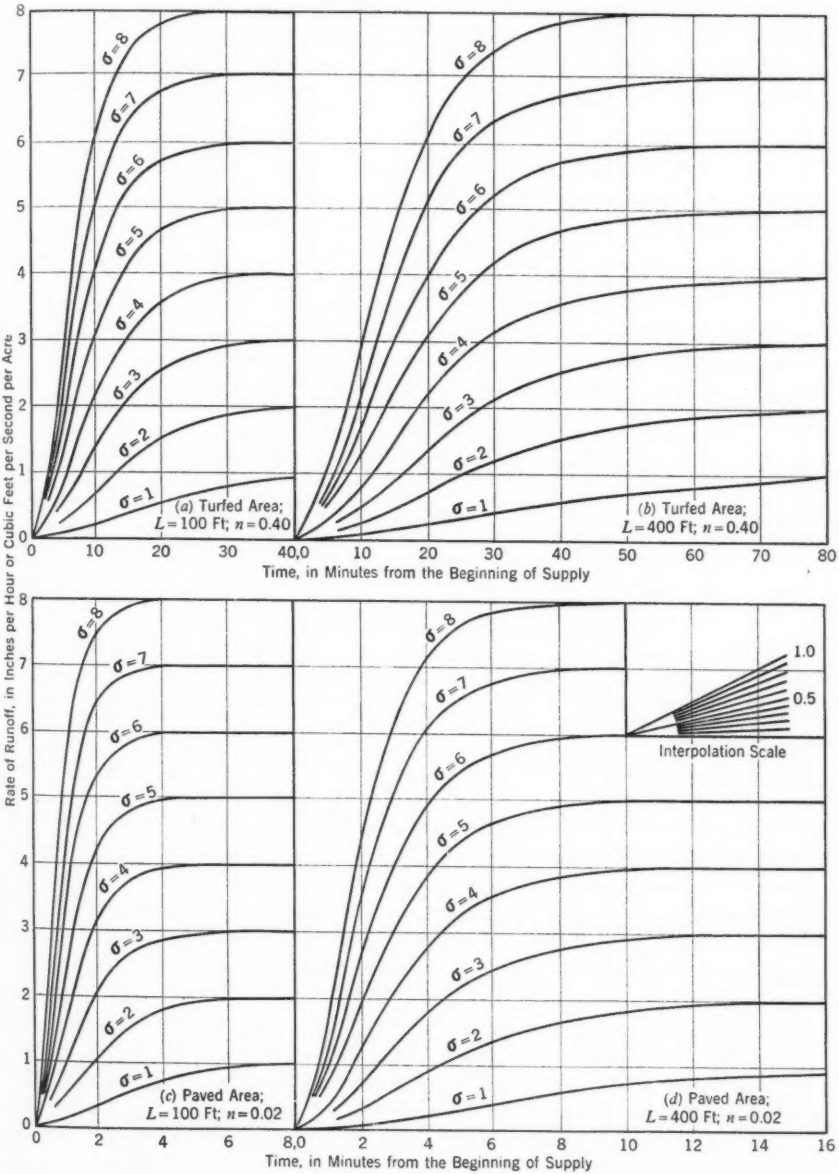


FIG. 16.—OVERLAND FLOW RATES VERSUS DURATION OF SUPPLY FOR PAVED AND TURFED AREAS WITH MAXIMUM SURFACE SLOPE, S, OF 1%

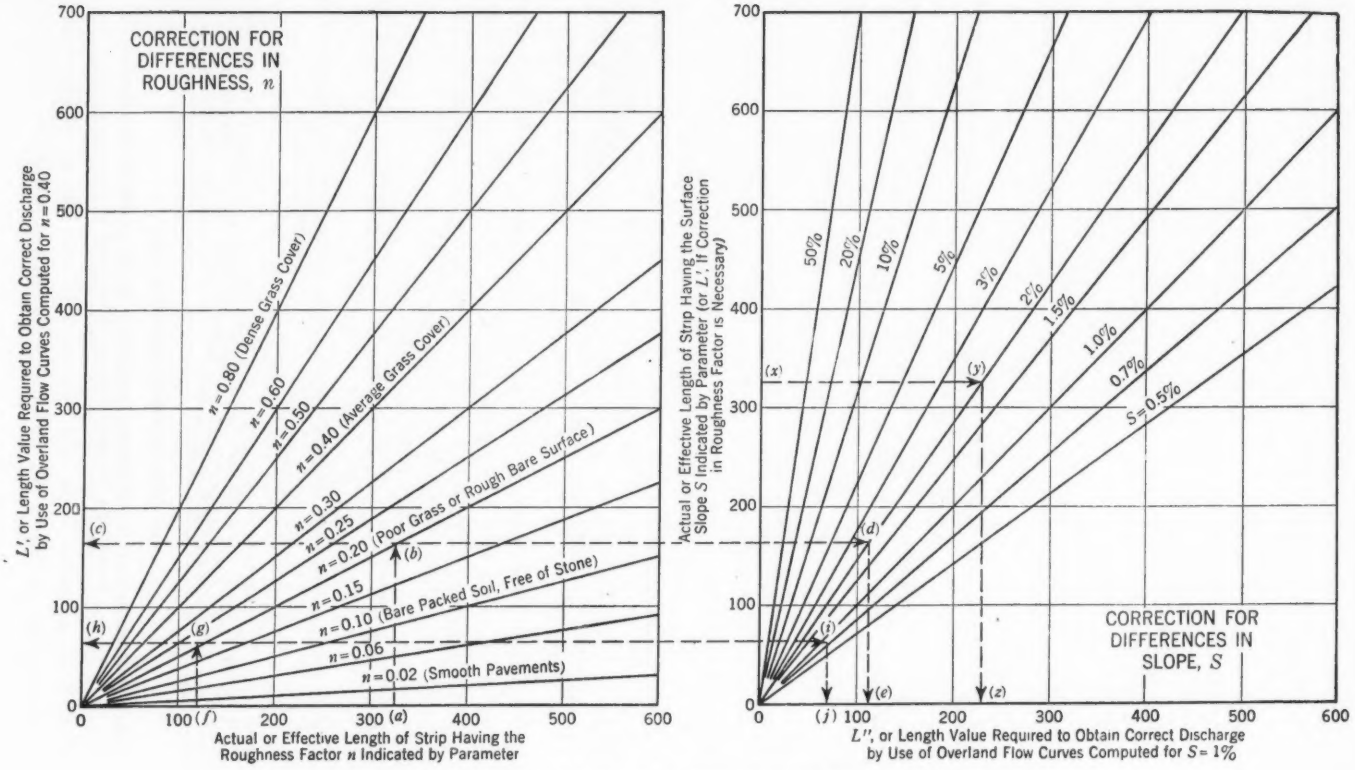


FIG. 17.—MODIFICATION IN L REQUIRED TO COMPENSATE FOR DIFFERENCES IN n AND S

rate of overland flow 25 min after the beginning of supply would be 1.7 in. per hr. Similarly, the rate of overland flow 40 min after the beginning of supply would be 2.0 in. per hr. Hydrographs of runoff corresponding to any selected duration and rate of rainfall excess may be computed from the overland flow rates indicated in Fig. 16 by the procedure shown in Fig. 18. The overland

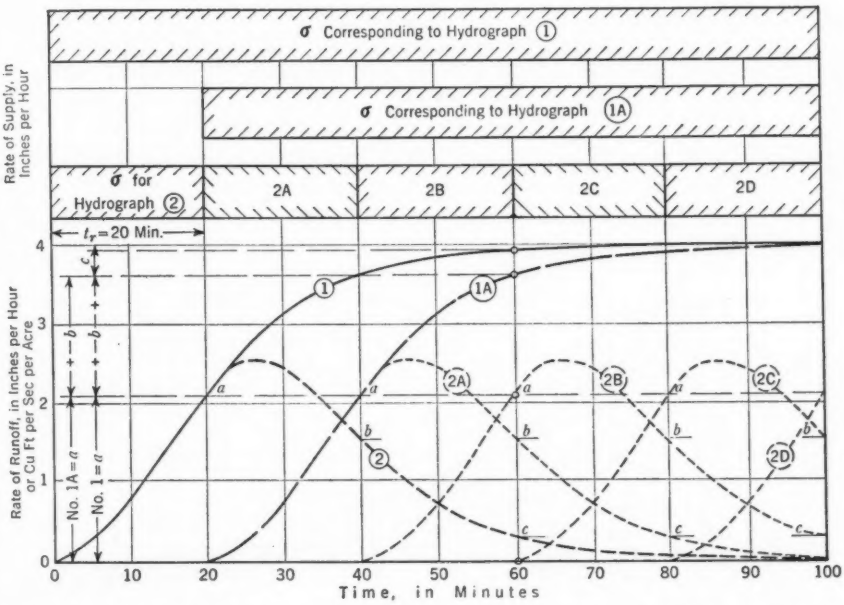


FIG. 18.—COMPUTATION OF HYDROGRAPH TO REPRESENT RUNOFF FROM A SUPPLY OF SPECIFIED DURATION

flow curve No. 1, of Fig. 18, represents the runoff hydrograph from an elemental strip of turf 400 ft in length assuming a continuous and uniform rate of supply (σ) of 4 in. per hr. The ordinates of the overland flow curve can be considered to be the sum of the corresponding ordinates of a series of identical runoff hydrographs (2, 2A, 2B, 2C, etc., not as yet determined) resulting from a continuous series of short rainfall periods of specified duration—for example, 20 min. Overland flow curve No. 1A is the same curve as No. 1, but is translated horizontally a time distance equal to the selected duration of 20 min, and represents the runoff for a continuous and uniform rate of supply of 4 in. per hr beginning 20 min after that of curve No. 1. Hydrograph No. 2 is obtained by subtracting the ordinates of No. 1A from the ordinates of No. 1 and approximately represents the runoff hydrograph to be expected from a supply rate of 4 in. per hr for a duration of 20 min. This is verified readily by a consideration of the composition of overland flow curves No. 1 and No. 1A at any particular time, say, for example, at 60 min. Hydrograph No. 2 (identical to 2A, 2B, etc.) must be such that the sum of the first 20-min value, *a*, of No. 2B, plus the second 20-min value, *b*, of No. 2A, plus the third 20-min value, *c*, of No. 2, equals the

ordinate of No. 1 at 60 min. However, No. 1A at 60 min equals No. 1 at 40 min by construction, and No. 1 at 40 min is equal to ordinate a of No. 2A plus b of No. 2. Since hydrographs 2, 2A, 2B, etc., are identical, all a ordinates are equal to each other as are the b, c, etc., ordinates. Thus at the time of 60 min No. 1 = $a + b + c$ and No. 1A = $a + b$. It follows that the ordinates of No. 1 minus the ordinates of No. 1A equal c, which is the 60-min value of hydrograph No. 2.

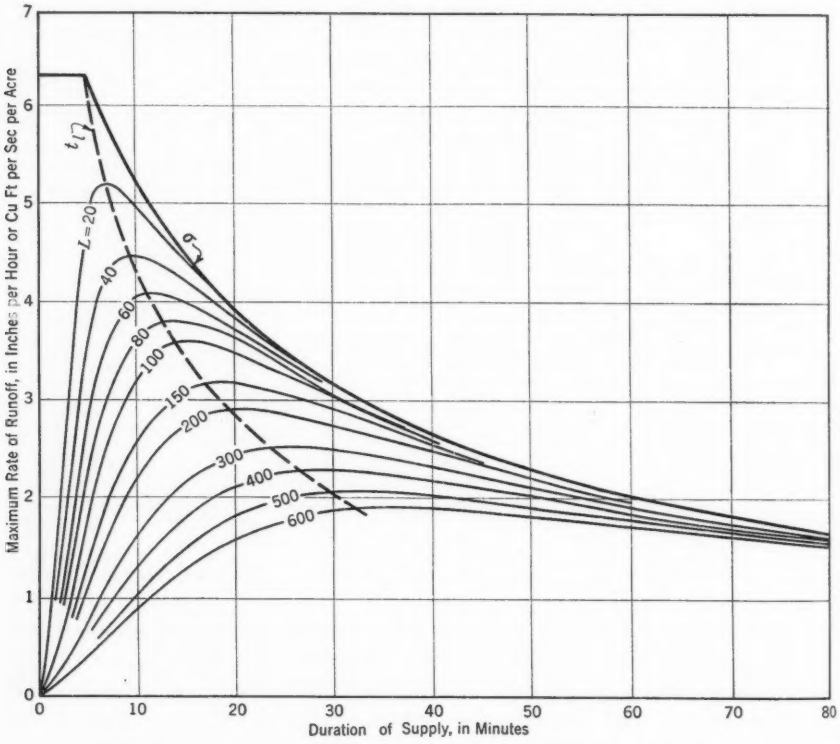


FIG. 19.—SUPPLY CURVE NO. 2.0; RATES OF OVERLAND FLOW CORRESPONDING TO STANDARD SUPPLY CURVES; $n = 0.40$, AND $S = 1\%$

The curves shown in Fig. 19 have been included for the purpose of demonstrating the relationship between the maximum rate and critical time of runoff (t_l) for various lengths L of overland flow and a specific supply curve. The curves were obtained by computing the rates of discharge at successive time intervals that would result from the various rates of supply σ, corresponding to standard supply curve No. 2.0 by application of the overland flow curves shown in Fig. 16. The procedure is illustrated by the sample computations in Table 2. To illustrate the procedure, durations of supply of 12, 20, and 35 min were selected and have been marked by dotted lines in Fig. 20. The rates of supply (scaled from No. 2.0 curve) for durations of 12, 20, and 35 min are 4.83, 3.85,

and 2.84 in. per hr, respectively (see Col. 2, Table 2, and points on curve A, Fig. 20). The overland flow rate was then scaled by interpolation between $\sigma = 4$ and $\sigma = 5$ (from curves similar to those shown in Fig. 16) for a length of 200 ft, a duration of 12 min, and a supply rate of 4.83 in. per hr, and determined to be 2.41 in. per hr (see Col. 6, Table 2, and the point on curve B, Fig. 20). The foregoing runoff values are accepted as the maximum under the assumption that surface storage is negligible. Actually the maximum runoff would occur a short time after the rainfall-excess ceases. For all practical purposes, however, the maximum rate of overland flow can be assumed to occur approximately at a time coincident with the ending of the rainfall-excess.

TABLE 2.—RATES OF OVERLAND FLOW CORRESPONDING TO INTENSITIES AND DURATION OF SUPPLY REPRESENTED BY STANDARD SUPPLY CURVE NO. 2.0 IN FIG. 15

($n = 0.40$, $S = 1\%$)

Duration of supply, in min	Rate of supply, in in. per hr[a]	Rate of Overland Flow[b] Corresponding to Durations Shown in Col. 1 and Rates of Supply Given in Col. 2						
		$L = 20$ ft	$L = 60$ ft	$L = 100$ ft	$L = 200$ ft	$L = 300$ ft	$L = 400$ ft	$L = 600$ ft
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
3	6.30	2.68	1.12	0.75	0.39	0.25	0.22	0.13
5	6.30	4.74	2.59	1.76	0.96	0.64	0.52	0.33
7	5.81	5.16	3.41	2.55	1.54	1.12	0.83	0.58
9	5.35	5.06	3.84	3.02	1.94	1.42	1.10	0.76
12	4.83	4.75	4.07	3.43	2.41	1.80	1.49	1.02
15	4.41	4.39	4.02	3.59	2.70	2.12	1.76	1.26
20	3.85	3.85	3.70	3.46	2.86	2.39	2.05	1.55
25	3.44	....	3.38	3.27	2.85	2.49	2.20	1.73
30	3.12	....	3.12	3.02	2.77	2.49	2.25	1.85
35	2.84	....	....	2.81	2.60	2.39	2.20	1.86
40	2.62	....	....	2.62	2.48	2.32	2.15	1.86
45	2.43	....	....	....	2.32	2.21	2.09	1.86
50	2.27	....	....	....	2.20	2.11	2.00	1.82
60	2.00	....	....	....	1.96	1.92	1.86	1.72
80	1.62	....	....	....	1.60	1.59	1.56	1.50
100	1.38	....	....	....	1.38	1.35	1.33	1.28
120	1.16	....	....	....	....	1.16	1.16	1.12
t_c[c]	....	7	11	14	20	24	28	32

[a] Scaled from curve No. 2.0, Fig. 15. [b] As scaled from diagrams similar to those shown in Fig. 16. [c] Critical duration of supply interpolated from Fig. 19.

Attention is directed to the fact that the curves in Fig. 19 are not hydrographs for any specific design storm, but represent the peak rates of runoff from individual storm events of various durations, all of which have the same average frequency of occurrence. The duration of supply corresponding to the maximum discharge for a particular standard supply curve and value of L in Fig. 19 is indicated by the broken line designated as t_l, the "critical duration of supply," defined as "the duration of rainfall-excess (rate of supply) corresponding to a given standard supply curve that would produce the maximum rate of outflow from a given drainage area, taking into account surface detention and surface runoff characteristics."

The curves in Fig. 16 represent the rate of overland flow that would result from various continuous rates of supply of uniform intensity. The maximum

rate of runoff for the respective uniform supply rates occurs at a time ordinarily referred to as the "time of concentration," when the rate of runoff becomes equal to the rate of supply. Curves A, B, and C of Fig. 20 are defined as follows:

A.—Standard supply curve No. 2 (Fig. 15 and Col. 2, Table 2);

B.—Maximum rates of overland flow corresponding to specific intensities and durations of supply represented by the standard supply curve No. 2 (Fig. 19 and Col. 6, Table 2); and

C.—Rate of overland flow for turfed areas corresponding to a uniform continuous supply rate of 2 in. per hr.

It will be observed that the rate of runoff represented by curve C, Fig. 20, approaches quite closely the rate of supply of 2 in. per hr at a time 60 min after

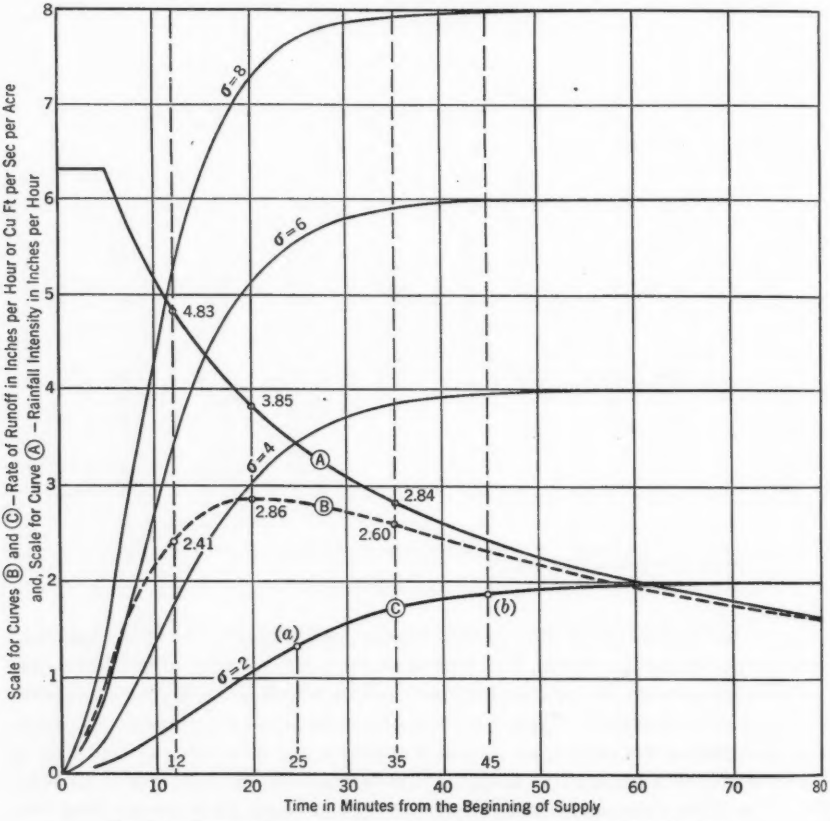


FIG. 20.—CRITICAL RATES OF OVERLAND FLOW VERSUS DURATION OF SUPPLY (L = 200 FT, n = 0.40, AND S = 1%)

the beginning of supply. Curve B, Fig. 20, represents the maximum rates of overland flow corresponding to specific intensities and durations of supply selected from curve A. Curve B is approximately equal to the rate of supply

of 2 in. per hr as represented by curve A at a duration of 60 min and is equal to curve C at 60 min. However, further inspection of curve B shows that a discharge greater than that represented by curve C at 60 min will result from durations of supply having the same average frequency (curve A) but higher intensities and shorter durations. In other words, under the assumed conditions, the maximum rate of overland flow for intensities and durations of supply represented by curve A and a length of 200 ft occurs with a supply rate of 3.85 in. per hr and a duration of supply of 20 min (see curves A and B, Fig. 20).

Drain-Inlet Capacity Versus Surface-Storage Relationships

If the rate of outflow from a drainage area is limited by the capacity of a drain serving the area, runoff occurring at rates exceeding the drain capacity must be stored temporarily. As soon as the rate of inflow into the ponding basin becomes less than the drain capacity, the accumulated storage may be drawn off at a rate equal to the difference between the drain capacity and the rate of inflow into the basin. The general relation between inflow, storage, and outflow is expressed by the equation:

$$\text{Outflow} = \text{inflow} \pm \text{storage} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The rate of outflow from a ponding basin is affected somewhat by the elevation of the water surface at the inlet to the drain serving the area, the rate of outflow increasing as the head on the inlet increases. However, because of the flat slopes of airfield areas, the surfaces of the storage ponds surrounding the drain inlets are usually very large in comparison with the depth of water at the inlets. Therefore, it is reasonable to assume that the rate of outflow through a particular drain inlet would be approximately constant as long as the rate of runoff and accumulated storage are sufficient to maintain the full discharge capacity of the drain inlet serving the particular area. It is also assumed that the rate of outflow would equal the rate of inflow into the pond until the full discharge capacity of the drain inlet is attained. To illustrate these assumptions reference is made to the curves shown in Fig. 21 for a length $L = 400$ ft, $S = 1\%$, $n = 0.40$, and a uniform "rate of supply" (σ) of 4 in. per hr for a duration of 20 min. Length L was chosen to represent the weighted mean distance from the limits of the drainage area to the average limits of the pond. Hydrographs 1, 2, and 3, Fig. 21, are:

1. Runoff that would have occurred if the given rate of supply (4 in. per hr) had continued indefinitely;
2. Runoff from a supply of 4 in. per hr for a duration of 20 min; and
3. Rate of outflow, assuming the inlet capacity as designated, and a constant head-discharge relation.

Hydrograph 3, Fig. 21, would represent the constant rate of outflow corresponding to the inflow hydrograph 2 if the drain-inlet capacity were assumed to be 1.25 cu ft per sec per acre of drainage area. The volume of storage above the outflow hydrograph 3 and below the inflow hydrograph 2 that would be accumulated at successive intervals of time under these conditions

is indicated by curve 4, Fig. 21. The maximum volume of storage that would accumulate under these particular conditions is 1,350 cu ft per acre of drainage area and the end of the accumulation period occurs approximately 43 min after the beginning of runoff.

Discharge-Storage Diagram

Hydrograph 2, Fig. 21, represents the rate of runoff from one acre of turf with a length of 400 ft, a coefficient n of 0.40, and a slope of 1%, assuming a uniform value of σ equal to 4 in. per hr for a duration of 20 min. A series of hydrographs similar to hydrograph 2 was computed for a "rate of supply" of

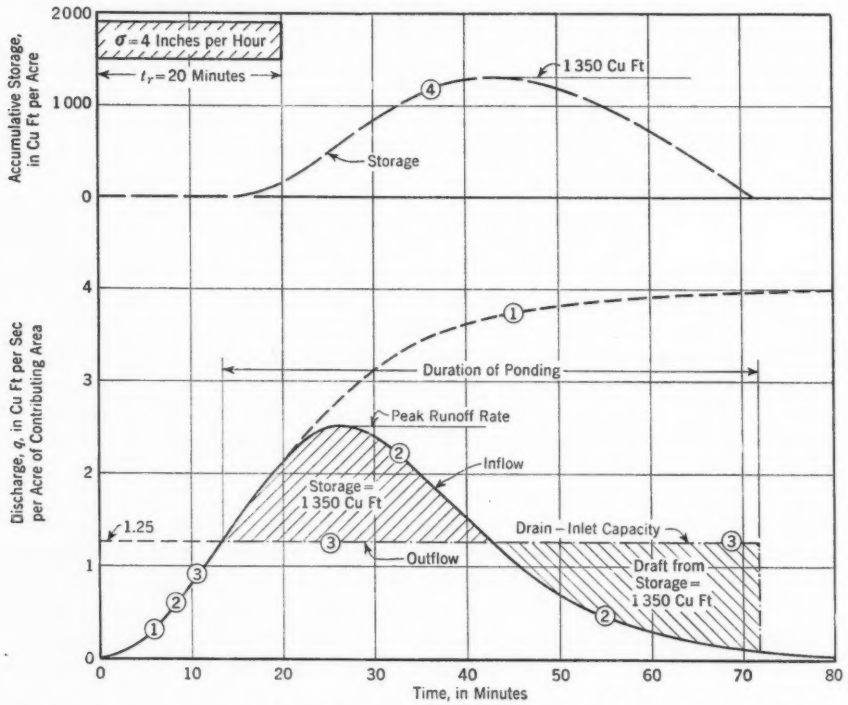


Fig. 21.—Computations for Storage Required with Selected Drain-Inlet Capacity to Provide for Runoff from 1 Acre of Turf; L = 400 Ft, n = 0.40, and S = 1%

4 in. per hr and for durations of supply of 10, 20, 30, 40, 50, and 60 min, successively. In each case the drainage area was taken as 1 acre and the "effective length" (L) as 400 ft. The volumes of runoff under the respective hydrographs and above abscissas representing discharge rates (drain-inlet capacity), similar to that of hydrograph 3 in Fig. 21, equal to 25%, 50%, and 75% of the peak ordinates thereof, were measured by planimeter. The volumes thus determined represent approximately the maximum quantities of water that would be stored temporarily in surface ponds if drain-inlet capacities equal to 25%, 50%, and 75% of the peak ordinates of the inflow hydrograph were adopted. The data

were plotted to obtain the series of curves in Fig. 22 for σ equal to 4 in. per hr. The same procedure was followed in developing a series of curves similar to that shown in Fig. 22 for "rates of supply" of 1, 2, 3, 4, 5, 6, 7, and 8 in. per hr assuming effective lengths (L) of 40, 100, 200, 300, 400, and 600 ft, successively. The foregoing procedure is one step in the preparation of "discharge versus storage diagrams" and Fig. 22 has been included to illustrate the step.

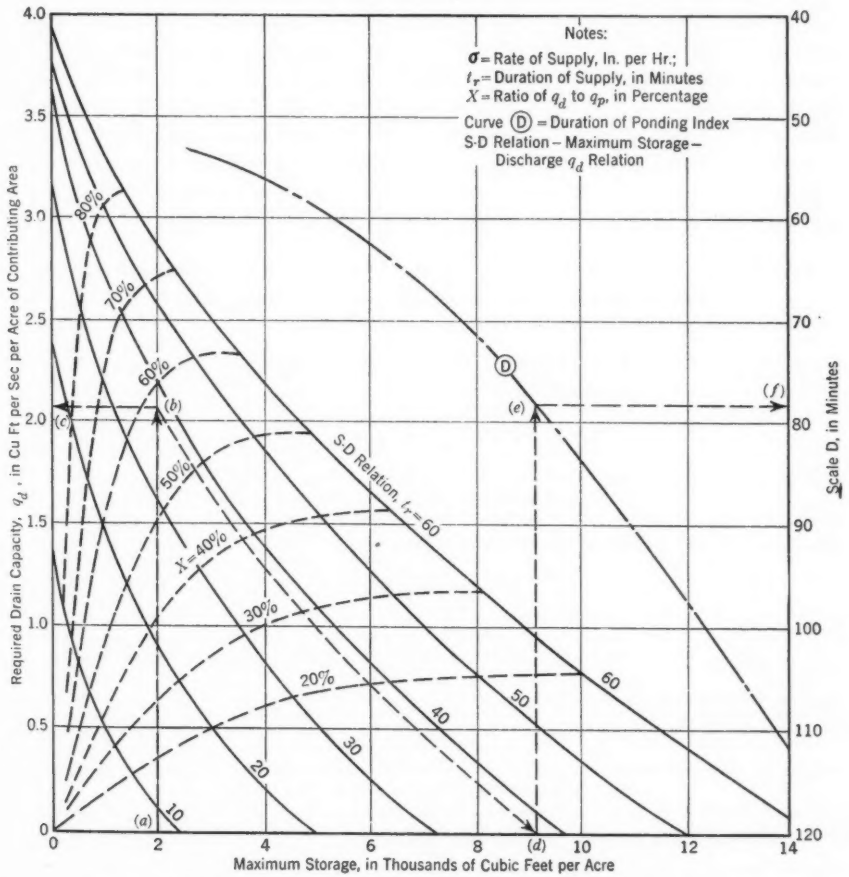


FIG. 22.—STORAGE-DISCHARGE RELATIONS FOR TURFED AREAS; L = 400 FT AND σ = 4

Inasmuch as the volume of storage accumulated in a ponding area is a function of both the rate and duration of supply, a "trial" computation procedure is necessary to determine the drain-inlet capacity required to limit storage to a specific volume, assuming a design storm criterion represented by one of the "standard rainfall intensity-duration curves" in Fig. 15. The computations involved in this procedure are illustrated in Table 3. The series of diagrams similar to Fig. 22 was used in computing the drain-inlet capacities

in Cols. 3 to 9, Table 3. "Rates of supply" (σ), corresponding to values used in preparing the diagrams in Fig. 22, were selected to avoid the need for interpolation (see Col. 1, Table 3). The durations of the supply rate corresponding to the "rates of supply" listed in Col. 1 were scaled from the "standard supply curves" in Fig. 15, and the values were placed in Col. 2, Table 3. The procedure followed in computing drain-inlet capacities required to limit surface storage to volumes indicated in the headings of Cols. 3 to 9, Table 3, assuming the various combinations of rate and duration of supply tabulated in Cols. 1 and 2, Table 3, may be illustrated by the following sample computation. Assume a turfed area for which $L = 400$ ft; $\sigma = 4$ in. per hr; $t_r = 38$ min, and a maximum permissible storage volume equal to 2,000 cu ft per acre of drainage area. The drain-inlet capacity required for the assumed condition would be determined by entering the diagram in Fig. 22 for $\sigma = 4$, at point a (storage 2,000), following a vertical line to point b ($t_r = 38$), and reading the drain-inlet capacity of 2.1 cu ft per sec per acre of drainage area at point c.

TABLE 3.—DRAIN-INLET CAPACITIES FOR VARIOUS SURFACE STORAGE VOLUMES TURFED AREA; $L = 400$ FT, $n = 0.40$, AND $S = 1\%$

Assumed rate of supply, σ, in in. per hr (Fig. 15)	Duration corresponding to assumed supply rate, in min (Fig. 15)	Assumed Maximum Surface Storage, in Cubic Feet Per Acre of Drainage Area						
		500	1,000	2,000	3,000	4,000	5,000	6,000
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
(*a*) Standard Supply Curve No. 1; See Fig. 15								
Drain-Inlet Capacity, q_d, in Cubic Feet Per Second Per Acre[a]:								
1	60	0.6	0.4	....	....	....	....	....
2	23	0.7	0.4	....	....	....	....	....
3	11	0.5	0.3	....	....	....	....	....
4	5	....	....	....	....	....	....	....
Critical,[b] q_d	....	0.7	0.4	....	....	....	....	....
Ratio $\frac{q_p}{q_d}$ (*X*) (%)	....	61	41	....	....	....	....	....
Duration, in Minutes:								
D-scale values		80	80	....	....	....	....	....
Maximum, ponding[b]		59	100	....	....	....	....	....
Critical supply[b]		33	38	....	....	....	....	....
(*b*) Standard Supply Curve No. 2; See Fig. 15								
Drain-Inlet Capacity, q_d, in Cubic Feet Per Second Per Acre[a]:								
1	164	0.9	0.8	0.7	0.6	....	....	....
2	60	1.5	1.3	1.0	0.7	....	....	....
3	32	1.8	1.5	1.0	0.6	....	....	....
4	19	1.7	1.4	0.8	0.4	....	....	....
5	11	1.4	1.0	0.5	....	....	....	....
6	6	1.0	0.6	0.1	....	....	....	....
Critical,[b] q_d	....	1.8	1.5	1.0	0.7	....	....	....
Ratio $\frac{q_p}{q_d}$ (*X*) (%)	....	80	64	57	40	....	....	....
Duration, in Minutes:								
D-scale values		75	75	118	118	....	....	....
Maximum, ponding[b]		35	53	99	150	....	....	....
Critical supply[b]		32	36	46	56	....	....	....

TABLE 3.—(*Continued*)

Assumed rate of supply, σ, in in. per hr (Fig. 15)	Duration corresponding to assumed supply rate, in min (Fig. 15)	Assumed Maximum Surface Storage, in Cubic Feet Per Acre of Drainage Area						
		500	1,000	2,000	3,000	4,000	5,000	6,000
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
(*c*) Standard Supply Curve No. 3; See Fig. 15								
Drain-Inlet Capacity, q_d, in Cubic Feet Per Second Per Acre[a]:								
2	112	1.3	1.7	1.5	1.4	1.2	1.1	....
3	60	2.6	2.3	1.9	1.6	1.3	1.1	....
4	38	2.9	2.6	2.1	1.7	1.3	1.0	....
5	24	3.0	2.6	1.9	1.4	1.0	0.7	....
6	15	2.7	2.1	1.4	0.9	0.5	0.1	....
7	10	2.3	1.8	1.0	0.5	0.1	....	....
8	6	1.6	1.3	0.6	0.2	....	....	....
Critical,[b] q_d	....	3.0	2.6	2.1	1.7	1.3	1.1	....
Ratio $\frac{q_p}{q_d}$ (X) (%)	....	76	74	59	47	46	....	....
Duration, in Minutes:								
D-scale values		58	78	78	78	115	....	....
Maximum, ponding[b]		32	42	62	89	126	....	....
Critical supply[b]		30	34	42	49	58	....	....
(*d*) Standard Supply Curve No. 4; See Fig. 15								
Drain-Inlet Capacity, q_d, in Cubic Feet Per Second Per Acre[a]:								
2	198	2.0	1.9	1.8	1.7	1.6	1.5	1.4
3	103	2.9	2.8	2.5	2.3	2.1	1.9	1.7
4	60	3.6	3.3	2.9	2.5	2.2	1.9	1.7
5	41	4.2	3.7	3.1	2.7	2.3	1.9	1.6
6	27	4.2	3.6	2.9	2.3	1.8	1.5	1.2
7	18	3.9	3.3	2.6	1.8	1.4	0.9	0.5
8	13	3.4	2.9	2.0	1.4	1.0	0.5	0.1
Critical,[b] q_d	....	4.2	3.7	3.1	2.7	2.3	1.9	1.7
Ratio $\frac{q_p}{q_d}$ (X) (%)	....	90	78	67	56	47	51	43
Duration, in Minutes:								
D-scale values		80	80	80	80	80	117	117
Maximum, ponding[b]		30	40	54	70	90	112	142
Critical supply[b]		....	33	38	46	53	60	67

[a] Drain-inlet capacity q_d in cubic feet per second per acre of drainage area required to limit storage to the assumed volumes. These values were computed with the assistance of curves similar to Fig. 22. [b] The critical value of q_d, the maximum duration of ponding, and the critical duration of supply were used in preparing diagrams A, B, and C, respectively, in Fig. 23.

As all rates of supply represented by a particular "standard supply curve" (Fig. 15) are assumed to occur with the same average frequency, the drain-inlet capacity required for a particular area should be adequate to limit the volume of storage to the maximum permissible quantity during all storms producing supply rates corresponding to the "standard supply curve" selected as the design criteria. Accordingly, only the "critical" drain-inlet capacities indicated in Table 3 are of prime concern in the determination of design capacities for storm drains. The relationships between "critical" drain-inlet capacities, maximum storage volumes, and "standard-supply-curve" numbers corresponding to those shown in Fig. 15 have been resolved into the convenient curves, an example of which is shown in Fig. 23, diagram A.

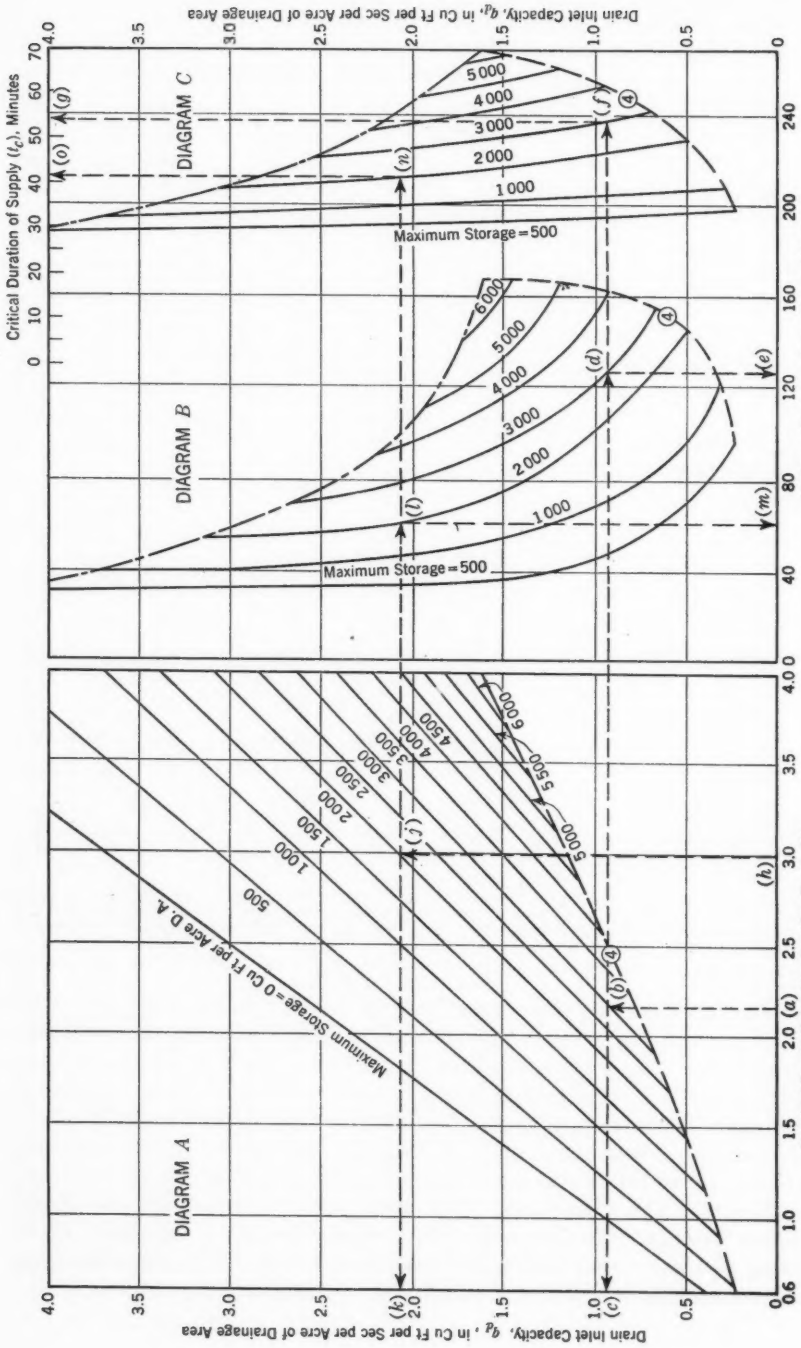


DIAGRAM A
DIAGRAM B
DIAGRAM C
Maximum Storage = 0 Cu Ft per Acre D. A
500
1 000
1 500
2 000
2 500
3 000
3 500
4 000
4 500
5 000
5 500
6 000
Maximum Storage = 500
Maximum Storage = 500
Drain Inlet Capacity, q_i, in Cu Ft per Sec per Acre of Drainage Area
Standard Supply Curve Number (See Figure 15)
Critical Duration of Supply (t_c), Minutes
Maximum Duration of Ponding, in Minutes
Drain Inlet Capacity, q_i, in Cu Ft per Sec per Acre of Drainage Area

FIG. 23.—DRAIN-INLET CAPACITY VERSUS MAXIMUM SURFACE STORAGE FOR TURFED AREAS; L = 400 FT, S = 1%, AND n = 0.40

The expression "duration of ponding" refers to a period beginning at the time the rate of storm runoff first exceeds the drain-inlet capacity and ending at the instant all the accumulated storage has been discharged (see Fig. 21). The duration of ponding corresponding to various combinations of drain-inlet capacity, permissible volume of surface storage, rate and duration of supply, and rate of runoff may be conveniently computed by use of charts similar to the one shown in Fig. 22 and the charts shown in Figs. 24 and 25.

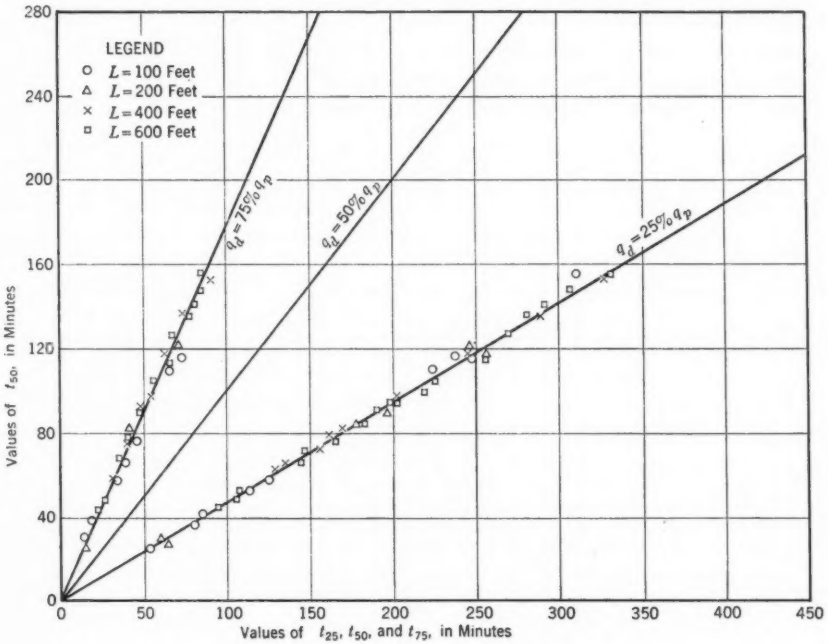


FIG. 24.—BASIC CORRELATION OF D_{50} TO D_{25} AND D_{75}

Figs. 22, 24, and 25 were included only to illustrate the manner in which the curves of diagrams A and B of Fig. 23 were derived; for example:

(*a*) The drain-inlet capacities equal to 25%, 50%, and 75% of the peak ordinates were assumed and the corresponding durations of ponding (t_{25}, t_{50}, and t_{75}) were computed by determining the length of time the drain-inlet capacity would have to remain at, say, t_{50}, in order for the accumulated volume of storage to be drawn off. This storage volume is illustrated clearly in Fig. 21.

(*b*) Values of t_{25} and t_{75}, computed in step (*a*), were plotted against corresponding values of t_{50} to determine average relationships for drain-inlet capacities equal to 25% and 75% of the peak rates of runoff for various sets of conditions (see Fig. 24). (Plotted points in Fig. 24 were computed from hydrographs similar to curve No. 2, Fig. 21, for various rates, durations of supply, and lengths *L*.)

FIG. 23.—DRAIN-INLET CAPACITY VERSUS MAXIMUM SURFACE STORAGE FOR TURFED AREAS; L = 400 FT.

(*c*) To facilitate interpolations, durations of ponding in minutes, scaled from Fig. 24, were plotted against drain-inlet capacity expressed as percentages of peak discharge (q_p) with values of t_{50} as parameters (see Fig. 25(*a*)).

(*d*) A working diagram (Fig. 25(*b*)) was prepared using data scaled from Fig. 25(*a*), with values of t_{50} ("scale *D*," Fig. 22) plotted against duration of ponding in minutes, with ratio (*X* of Fig. 22) of drain-inlet capacity to peak rate of runoff as parameters.

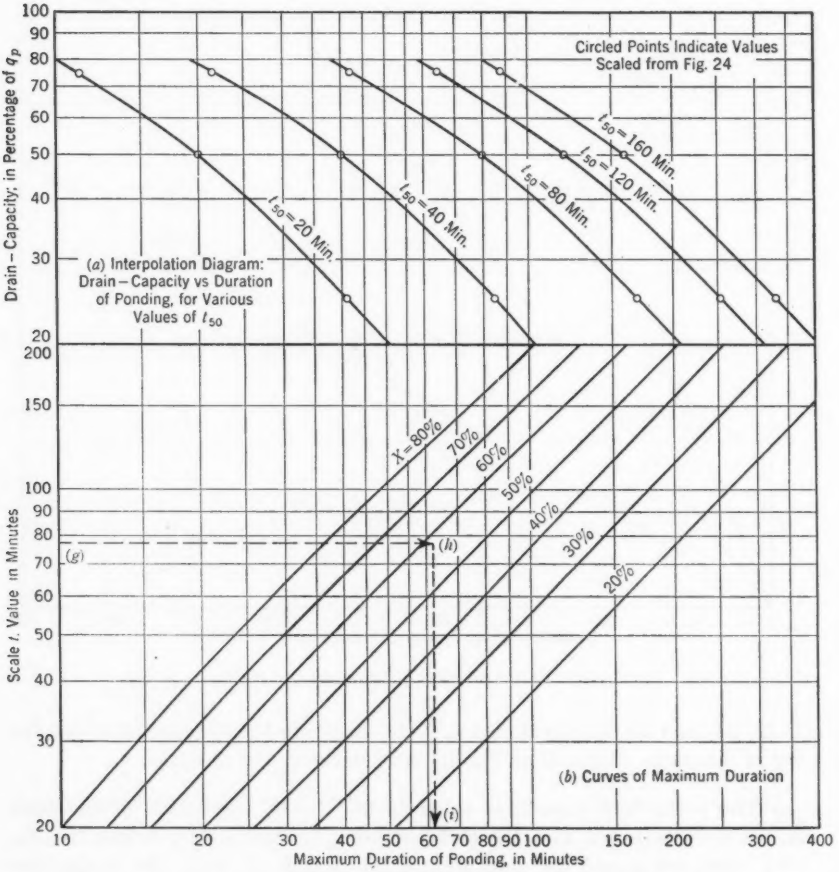


FIG. 25.—METHOD OF DETERMINING MAXIMUM DURATION OF PONDING

(*e*) Values of t_{50} were plotted in Fig. 22 against maximum storage, assuming zero drain-inlet capacity and various durations of supply to obtain the curves designated thereon by the letter *D*.

(*f*) The use of the curves shown in Figs. 22, 24, and 25 may be illustrated by continuing the example referred to previously with a length *L* of 400 ft, σ of 4 in. per hr, a duration of supply (t_r) of 38 min, and a maximum permissible

storage volume of 2,000 cu ft per acre of drainage area. Using these values a drain-inlet capacity of 2.1 cu ft per sec per acre of drainage area was determined from Fig. 22. By interpolation determine the value of the X-ratio (59%) corresponding to point b, and record the X-ratio for future use. Follow the "S-D relation" curve to a t_r of 38 min from point b to point d; then proceed vertically to point e and read a "scale D" value of 78 min at point f. Enter Fig. 25(*b*) at point g with a "scale D" value of 78 min; follow a horizontal line to a point corresponding to an X-ratio of 59%; and read a value for the maximum duration of ponding at point i of 62 min on the bottom scale.

(*g*) Results obtained by the procedure outlined in step (*f*), for the critical values shown in Table 3, are tabulated under the subdivisions in Table 3 for the various "supply curve numbers" (see footnote ([b]), Table 3).

Therefore, to determine the maximum drain-inlet capacity for various durations of supply (Col. 2, Table 3) and for various storages (Cols. 3 to 9, Table 3), the drain-inlet capacity (Cols. 3 to 9) was plotted against the duration of the supply rate in minutes (Col. 2) corresponding to the assumed "rate of supply" in inches per hour (Col. 1). The procedure may be illustrated by specific reference to Table 3: The drain-inlet capacities assuming a "standard supply curve No. 3," for example, for a storage volume of 2,000 cu ft per acre of drainage area are shown in Col. 5. These values were plotted against the duration of the supply rate in minutes indicated in Col. 2, Table 3. A critical (or maximum) drain-inlet capacity of 2.1 cu ft per sec per acre of drainage area was scaled from this curve at an approximate duration of supply of 42 min. Similarly, the critical drain-inlet capacity of 1.7 cu ft per sec per acre for a storage volume of 3,000 cu ft per acre was scaled from the curve plotted for this condition, at a duration of supply of 49 min. The critical drain-inlet capacities obtained by this procedure are tabulated under the subdivisions for the various "supply curve numbers" (see footnote ([b]), Table 3) and for each of the storage values indicated in the headings of Cols. 3 to 9, Table 3. The results thus presented in Table 3 were plotted in diagram C, Fig. 23 (length of 400 ft), against the corresponding storages and critical drain-inlet capacities. Similar computations were made for lengths of 40, 100, 200, 300, and 600 ft, respectively, the results of which are omitted from the paper.

The use of diagrams A, B, and C, Fig. 23, is illustrated by the following example:

Given.—(1) Length L of turfed area = 400 ft; (2) design storm criteria as represented by "standard supply curve No. 3.0" in Fig. 15; and (3) maximum permissible storage = 2,000 cu ft per acre of drainage area.

Solution.—Enter diagram A of Fig. 23 at point h (supply rate of 3.0) and follow a vertical line to point j (storage of 2,000); then follow a horizontal line left to the margin of diagram A to point k and read the required drain-inlet capacity of 2.1 cu ft per sec per acre of drainage area (see Col. 5, Table 3). The maximum duration of ponding is obtained by following a horizontal line from point j to the right to point l (storage 2,000) and reading a value of 62 min on the bottom scale of diagram B at point m (see Col. 5, Table 3). The critical duration of supply (t_c) is obtained by continuing the horizontal line from point l

to point n (storage 2,000) to read a value of 42 min at point o on the scale at the top of diagram C (see Col. 5, Table 3).

Effective Length of Drainage Area

As used in the Horton overland flow formulas, L refers to the over-all length of an elemental strip of turfed or paved surface in a direction parallel to the maximum slope. This definition of L must be liberalized somewhat when applied to an area in which a shallow large-area pond will be formed by storage of runoff during the design storm. In estimating the "effective length" for a drainage basin in which an appreciable quantity of temporary storage is permissible, the value of L adopted should be such that the rate of runoff indicated by the corresponding overland flow hydrograph would approximate the actual hydrograph of inflow into the ponding area. In such cases, the selected value of L may be considered as an index to be used in selecting the most applicable set of computation charts rather than as a definite dimension of the drainage area. With storage capacities ordinarily attainable and permissible in intermediate areas, the use of an approximate value of L will not introduce serious errors in the determination of the required drain capacities.

The value of L strictly applicable to a particular area varies as the size of the storage pond fluctuates during the course of storm runoff. As water accumulates in the relatively flat storage area during the process of storm runoff, the size of the pond increases rapidly, thus progressively reducing the distance from the edge of the pond to the outer limits of the drainage area. To simplify computation of the required drain-inlet capacity, it is desirable to select a value of L for the drainage area tributary to each inlet that most closely approximates the average during the period of critical runoff. In most cases it is satisfactory to estimate the "effective length" as the distance from the outer limits of the turfed drainage area to the mean boundary of the ponding area during the period of critical storm runoff. If the drain inlet is not located near the centroid of the turfed drainage area, the "effective length" should be estimated as the weighted average distance from the boundary of the turfed area to the mean limits of the ponding area corresponding to a depth equal to two thirds the maximum during the design storm.

As a general rule, ponding areas are located in turfed intermediate areas that are bordered by paved runways, taxiways, or paved aprons (see Fig. 26). Runoff from these paved areas must pass over turfed slopes to reach the ponding areas and drain inlets, and in so doing is subjected to somewhat the same retardance as the runoff that results from precipitation falling directly on the turfed area. Inasmuch as the time required for water to flow over paved areas is normally short (5 to 10 min), the length of the paved area can be disregarded or given very little weight in estimating the "effective length" (L) for a composite area. In exceptional cases where large paved landing mats or bare areas are involved it will be necessary to determine the modified effective length by means of the diagrams in Fig. 17.

Storm Drains

A sample layout of a storm drain system for a portion of an airfield is shown in Fig. 26. The inlets to the underground drains are located in such a manner

as to permit the temporary storage of substantial volumes of runoff during intense storms without interfering with normal use of runways, taxiways, and parking areas. To prevent the concentration of an excessive quantity of storage in the lower parts of the field, the intermediate areas were assumed to be graded in such a manner as to effect drainage toward the several drain inlets and to form shallow, large-area storage basins around each inlet. By providing storage ponds around several inlets a relatively large volume of water can be stored without causing excessive depths of ponding in any area during the design storm.

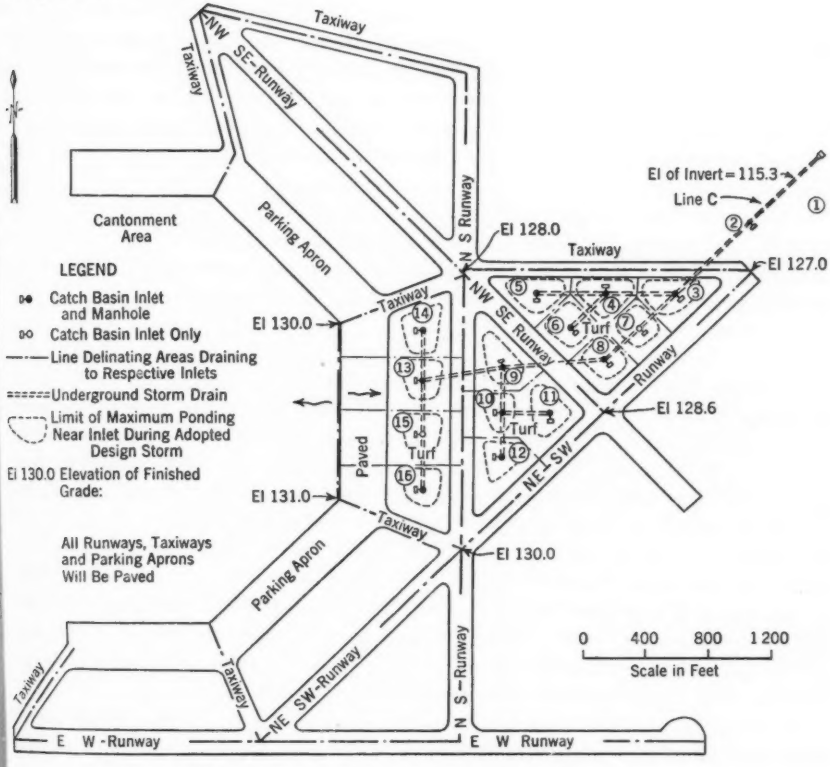


FIG. 26.—LAYOUT OF PRIMARY STORM DRAIN SYSTEM

The location of drain inlets must be carefully related to the grading plan for the intermediate area if ponding areas having sufficient storage capacity to effect important reductions in required drain-inlet capacities are to be provided without creating objectionable flooding conditions or undesirable rates of change in surface grades. The elevation of the various inlets will be determined by their location with respect to runways, aprons, etc., and the limiting surface grades applicable to the area. The distance between inlets will be determined by the elevation of the respective inlets with relation to controlling elevations of runways, aprons, etc., the general slope of the field, and the limiting grades pre-

scribed. For major airfields a maximum effective length of overland flow in excess of 400 ft (corrected to a slope of 1% and a roughness factor of 0.40) should not be permitted in areas adjacent to runways and aprons, and 600 ft in all-over fields.

The storage capacity that can be provided in intermediate areas without exceeding limiting grades in any direction is determined partly by the general slope of the field and partly by the location of the drain inlets with respect to runways, taxiways, and aprons. To estimate the maximum volume of storage permissible in a particular area, the maximum elevations of ponding allowable therein should be estimated by inspection of cross-sectional profiles, and the limits of the ponding area should be sketched on a map showing the drain-inlet layout. The surface areas of the various ponding areas may be measured by planimeter. Ordinarily, the storage capacity, in cubic feet below the elevation of maximum ponding, may be computed as one third of the maximum depth of ponding at the inlet, in feet, multiplied by the surface area of the pond, in square feet.

In general, the maximum depth of ponding in intermediate areas during the design storm should not exceed approximately one foot at the inlets if the intermediate areas are to be used for landing purposes and the upper limits of the storage ponds should not approach within 75 ft of runway pavements.

Minimum Drain Capacities

In some instances it would be feasible to provide surface ponds of sufficient capacity to store practically the entire runoff from storms less than a few hours in duration, without exceeding the maximum limits of the ponding areas. However, if the maximum permissible volume of storage were utilized in such cases, the drain-inlet capacities provided would be very small and the corresponding period required to remove the stored waters after each storm would be excessive. Furthermore, flooding of the lower parts of the ponding area would be relatively frequent; it may be practically continuous for several days as the result of a series of comparatively moderate storms. Although the area subjected to frequent and prolonged flooding may constitute a small proportion of the maximum permissible ponding area, the creation of unstable soil conditions in such areas would be objectionable in many cases.

Curve No. 4 in diagram A of Fig. 23 represents the minimum drain-inlet capacities that are considered desirable, regardless of the volume of storage capacity that may be permitted without the depth or area of flooding exceeding allowable limits. The drain-inlet capacities represented by curve No. 4, diagram A, Fig. 23, in cubic feet per second per acre of drainage area are equal to the rates of supply corresponding to a duration of 4 hr on the respective "standard supply curves" given in Fig. 15. If a drain-inlet capacity indicated by curve No. 4 is adopted in a particular case, it may be expected that some storage in the ponding basin will result during all storms less than 4 hr in duration that produce supply rates corresponding to the given "standard supply curve." The proper criteria to be followed in estimating minimum drain-inlet capacities depend largely upon the characteristics of the soil involved and the use that is to be made of the intermediate areas.

Computations of Storm Drain Capacities, Assuming Supplemental Ponding

The principal procedures involved in the determination of drain-inlet capacities required to limit storage in individual ponding areas to permissible volumes are shown by sample computations presented in Tables 4(*a*) and 5(*a*) for line C in the sample layout of Fig. 26. Assuming the project to be in north central Alabama, the following summarizes briefly the successive steps to be followed:

(*a*) Select a design-storm frequency (say of 2.2 in. per hr) approximately equivalent to a 5-yr frequency in Fig. 13(*d*).

(*b*) Estimate infiltration capacity and determine "standard supply curves" applicable to paved and unpaved areas. The infiltration capacity for a 1-hr rainfall for turfed areas in this case is assumed to be 0.6 in. per hr. The infiltration capacity for paved areas is zero; therefore the "standard supply curves" for paved and unpaved areas are Nos. 2.2 and 1.6, respectively (see Fig. 15).

(*c*) Compute "weighted standard supply curves" for composite drainage areas tributary to the respective drain inlets (see Cols. 15 to 19, Table 4(*a*)).

(*d*) By use of curves such as Fig. 23, compute the drain-inlet capacities required to limit storage above the respective inlets to permissible volumes (see sample computation in Table 4(*a*)).

In general, the methods used in current practice to design city storm sewers are applicable to the determination of pipe sizes for airfield storm drains. It is desirable to maintain the hydraulic grade line as near as practicable to the top of the drain pipe during the design storm. A minimum velocity of 2.5 ft per sec should be maintained in so far as practicable to minimize the deposition of sediment in the drains. Where ponding is contemplated the design of the storm drains should be reviewed, assuming the hydraulic gradient at the top of each inlet, to investigate the possibility of excessive flooding at any of the low inlets. If such conditions exist, the discharge from the inlets contributing to the excessive flooding should be regulated either by an orifice control in the inlet or by designing the inlet connection pipe for the design discharge reduced by ponding.

Computation of Storm Drain Capacity, Assuming No Supplementary Ponding

In certain instances it may be advisable to provide storm drains having sufficient capacities to remove runoff from the design storm without temporary ponding in the areas surrounding the drain inlets. To accomplish this purpose, drain-inlet capacities equal to the maximum rates of design storm runoff from areas tributary to respective inlets are required. An accurate determination of drain-inlet capacities necessary to remove design storm runoff from several inlet drainage areas without ponding entails a rather tedious procedure. Similar computations and a general consideration of the objectives involved indicate that the simple approximate method illustrated in Tables 4(*b*) and 5(*b*)

TABLE 4.—Drain-Inlet Capacities; Drainage Line C, Fig. [illegible]SING

Inlet No.	Drainage Area, in Acres				Permissible Ponding				Average		Length L [illegible]	
	Paved; n =0.02	Unpaved		Total	Depth at inlet; ft	Pond area; 1,000 sq ft	Volume, 1,000 cu ft	Volumes, cu ft per acre	Roughness factor n	Slope S; %	Actual, or effective	Equiva[illegible] L, fo[illegible] n =0.4 S =1[illegible]
		Bare; n =	Turf; n =0.4									
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13) [illegible]
												(*a*) Requirements for Limiting [illegible]DING
16	3.44	....	3.44	6.88	0.9	44	13	1,900	0.21	1.0	125	7[illegible]00
15	3.11	....	3.20	6.31	0.9	55	16	2,500	0.21	0.8	120	7[illegible]00
14	3.20	....	3.53	6.73	1.2	62	25	3,700	0.22	0.9	120	7[illegible]00
13	3.11	....	3.20	6.31	0.9	55	16	2,500	0.21	0.8	120	7[illegible]00
12	2.27	....	3.20	5.47	0.9	44	13	2,400	0.24	0.7	120	8[illegible]00
11	1.76	....	3.28	5.04	1.0	62	21	4,200	0.27	1.0	100	7[illegible]00
10	0.53	....	2.02	2.55	0.9	37	11	4,300	0.32	0.8	80	7[illegible]00
9	2.44	....	3.28	5.72	1.0	55	18	3,200	0.24	0.8	120	7[illegible]00
8	1.26	....	2.27	3.53	0.9	55	16	4,500	0.26	0.8	100	7[illegible]00
7	0.55	....	2.35	2.90	0.9	48	14	4,800	0.33	0.8	75	7[illegible]00
6	0.52	....	2.44	2.96	1.0	55	18	6,100	0.33	0.9	75	7[illegible]00
5	1.60	....	2.44	4.04	1.0	62	21	5,200	0.25	0.6	100	8[illegible]00
4	0.50	....	2.18	2.68	1.0	59	20	7,500	0.33	0.6	50	5[illegible]40
3	1.68	....	2.69	4.37	1.0	66	22	5,000	0.25	0.8	75	5[illegible]40
												(*b*) Requirements for Removing the Design-S[illegible]OFF V
16	3.44	....	3.44	6.88	....	....	....	0	0.21	1.0	220	11[illegible]15
15	3.11	....	3.20	6.31	....	....	....	0	0.21	0.8	200	12[illegible]20
14	3.20	....	3.53	6.73	....	....	....	0	0.22	0.9	200	11[illegible]15
13	3.11	....	3.20	6.31	....	....	....	0	0.21	0.8	200	12[illegible]20
12	2.27	....	3.20	5.47	....	....	....	0	0.24	0.7	200	14[illegible]40
11	1.76	....	3.28	5.04	....	....	....	0	0.27	1.0	180	12[illegible]20
10	0.53	....	2.02	2.55	....	....	....	0	0.32	0.8	160	14[illegible]45
9	2.44	....	3.28	5.72	....	....	....	0	0.24	0.8	200	13[illegible]35
8	1.26	....	2.27	3.53	....	....	....	0	0.28	0.8	170	13[illegible]35
7	0.55	....	2.35	2.90	....	....	....	0	0.33	0.8	160	14[illegible]45
6	0.52	....	2.44	2.96	....	....	....	0	0.33	0.9	160	13[illegible]35
5	1.60	....	2.44	4.04	....	....	....	0	0.25	0.6	180	16[illegible]45
4	0.50	....	2.18	2.68	....	....	....	0	0.33	0.6	120	13[illegible]20
3	1.68	....	2.69	4.37	....	....	....	0	0.25	0.8	160	11[illegible]15

[a] Value of L used for selecting the proper diagram. [b] This column is not required when appreciable po[illegible] 9) is [illegible] minimum capacity criteria by curves of the type of No. 4, Fig. 23; therefore, actual volumes and depths of st[illegible]g the Cols. 4 to 10 for computations of t_c in Table 4(*b*).

FIG. SING CURVES NOS. 2.2 FOR PAVED AREAS AND 1.6 FOR TURFED AREAS)

NGTH L, ET		SUPPLY CURVE NO. TIMES DRAINAGE AREA				Weighted curve, Col. 18 ÷ Col. 5	DRAIN-INLET CAPACITY			CRITICAL CONTRIBUTION TO SYSTEM			Inlet No.
Equiva- L, fo n =0.4 S =1	ma- mum ues[a]	Paved areas, Col. 2	Unpaved Bare, Col. 3	Unpaved Turf, Col. 4	Total, Col. 5		t_c (min)[b]	q_d (cu ft per sec)[c]	Q_d, Col. 21 × Col. 5	t_c (min)[e]	q_d (cu ft per sec)[b]	Q_d, Col. 24 × Col. 5	
(13)	14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(1)
MITING		DING TO PERMISSIBLE VOLUMES											
70	100	7.6	....	5.5	13.1	1.9	....	1.4	9.6	....	1.4	....	16
70	100	6.8	....	5.1	11.9	1.9	....	1.1	6.9	....	1.1	....	15
70	100	7.0	....	5.7	12.7	1.9	....	0.7[d]	4.7	....	0.7[d]	....	14
70	100	6.8	....	5.1	11.9	1.9	....	1.1	6.9	....	1.1	....	13
85	100	5.0	....	5.1	10.1	1.8	....	1.0	5.5	....	1.0	....	12
70	100	3.9	....	5.3	9.2	1.8	....	0.6[d]	3.0	....	0.6[d]	....	11
70	100	1.2	....	3.2	4.4	1.7	....	0.6[d]	1.5	....	0.6[d]	....	10
75	100	5.4	....	5.3	10.7	1.9	....	0.8	4.6	....	0.8	....	9
75	100	2.8	....	3.6	6.4	1.8	....	0.6[d]	2.1	....	0.6[d]	....	8
70	100	1.2	....	3.8	5.0	1.7	....	0.6[d]	1.7	....	0.6[d]	....	7
70	100	1.1	....	3.9	5.0	1.7	....	0.6[d]	1.8	....	0.6[d]	....	6
85	100	3.5	....	3.9	7.4	1.8	....	0.6[d]	2.4	....	0.6[d]	....	5
55	60	1.1	....	3.5	4.6	1.7	....	0.6[d]	1.6	....	0.6[d]	....	4
55	60	3.7	....	4.3	8.0	1.8	....	0.6[d]	2.6	....	0.6[d]	....	3
ESIGN-S		OFF WITHOUT TEMPORARY PONDING (ALTERNATE DESIGN)											
11	115	7.6	....	5.5	13.1	1.9	15	3.2	22.0	20	3.2	22.0	
										25	3.0	20.7	16
										30	2.9	20.0	
12	120	6.8	....	5.1	11.9	1.9	15	3.2	20.2	20	3.1	19.6	
										25	3.0	18.9	15
										30	2.9	18.3	
11	115	7.0	....	5.7	12.7	1.9	15	3.2	21.5	20	3.2	21.5	
										25	3.0	20.2	14
										30	2.9	19.5	
12	120	6.8	....	5.1	11.9	1.9	15	3.2	20.2	20	3.1	19.6	
										25	3.0	18.9	13
										30	2.9	18.3	
14	140	5.0	....	5.1	10.1	1.8	16	2.8	15.3	20	2.9	15.9	
										25	2.8	15.3	12
										30	2.6	14.2	
12	120	3.9	....	5.3	9.2	1.8	15	3.1	15.6	20	3.0	15.1	
										25	2.9	14.6	11
										30	2.7	13.6	
)	14	1.2	....	3.2	4.4	1.7	17	2.6	6.6	20	2.7	6.9	
										25	2.6	6.6	10
										30	2.5	6.4	
)	13	5.4	....	5.3	10.7	1.9	16	3.1	17.7	20	3.1	17.7	
										25	3.0	17.2	9
										30	2.8	16.0	
)	13	2.8	....	3.6	6.4	1.8	16	2.9	10.2	25	2.8	9.9	
										30	2.7	9.5	8
)	14	1.2	....	3.8	5.0	1.7	17	2.6	7.5	25	2.6	7.5	
										30	2.5	7.2	7
0	13	1.1	....	3.9	5.0	1.7	16	2.7	8.0	20	2.8	8.3	
										30	2.5	7.4	6
0	14	3.5	....	3.9	7.4	1.8	17	2.8	11.3	20	2.8	11.3	
										30	2.6	10.5	5
0	13	1.1	....	3.5	4.6	1.7	16	2.8	7.5	20	2.8	7.5	
0	11	3.7	....	4.3	8.0	1.8	15	3.0	13.1	30	2.7	11.8	3
										30	2.5	6.7	4

eciable po 9) is permissible. [c] Cubic feet per second per acre of drainage area. [d] These values were determined by lepths of st the design storm would be somewhat less than the values indicated in Cols. 6 and 9. [e] See Table 5(*b*).

afford a satisfactory degree of accuracy. In order that the design capacities with and without temporary ponding of runoff may be compared for the same conditions, the drain-inlet layout presented in Fig. 26 and the design storm criteria used in the sample computation given in Table 4(*a*) were used in Tables 4(*b*) and 5(*b*). A comparison of Tables 5(*a*) and 5(*b*) shows that the maximum flow is about 189 cu ft per sec without supplemental ponding as compared to 57 cu ft per sec with supplemental ponding.

TABLE 5.—DESIGN CAPACITY OF UNDERGROUND DRAINAGE; LINE C, FIG.

POINT OF DESIGN			CRITICAL RUNOFF TIME,[a] IN MINUTES						
Inlet, or junction No.	Distance, in ft, from:		Critical inlet No.	Critical inlet time, t_i	Assumed velocity in pipe, ft per sec	Drain Time		Approximate t_c, Col. 5 + Col. 8	Adopted t_c
	Main outlet	Preceding inlet				From preceding inlet	Accumulated total		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
(*a*) ASSUMING POND									
16	3,610	....	....	....	....	....	....	....	....
15	3,260	350	....	....	....	....	....	....	....
13	2,910	350	....	....	....	....	....	....	....
9	2,390	520	....	....	....	....	....	....	....
8	1,740	650	....	....	....	....	....	....	....
7	1,420	320	....	....	....	....	....	....	....
3	1,100	320	....	....	....	....	....	....	....
2	500	600	....	....	....	....	....	....	....
1	0	500	....	....	....	....	....	....	....
Branch Lines:									
14	3,260	0	....	....	....	....	....	....	....
12	2,990	0	....	....	....	....	....	....	....
11	2,990	0	....	....	....	....	....	....	....
10	2,690	300	....	....	....	....	....	....	....
6	1,860	0	....	....	....	....	....	....	....
5	1,980	0	....	....	....	....	....	....	....
4	1,540	440	....	....	....	....	....	....	....
(*b*) ASSUMING									
16	3,610	....	16	15	....	0	0	15	
15	3,260	350	16	15	3.5	2	2	17	
13	2,910	350	16	15	3.0	2	4	19	
9	2,390	520	16	15	3.0	3	7	22	
8	1,740	650	16	15	4.0	3	10	25	
7	1,420	320	16	15	3.0	2	12	27	
3	1,100	320	16	15	3.0	2	14	29	
2	500	600	16	15	4.0	2	16	31	
1	0	500	16	15	....	....	....	....	....
Branch Lines:									
14	3,260	0	14	15	....	0	0	15	
12	2,990	0	12	16	....	0	0	16	
11	2,990	0	11	15	....	0	0	15	
10	2,690	300	12	17	3.0	2	2	19	
6	1,860	0	6	16	....	0	0	16	
5	1,980	0	5	17	....	0	0	17	
4	1,540	440	5	16	2.5	3	3	19	

[a] Critical time required to produce the maximum flow in the underground drain. [b] Table 4(*a*) contains disc

A comparison of the pipe sizes required for drainage systems utilizing both instantaneous runoff and temporary ponding discloses that, in general, the system contemplating ponding utilizes approximately one third the discharge

capacity necessary for the instantaneous runoff system. This apparent saving is somewhat tempered because of the slightly increased grading costs. However, it can be shown that a considerable saving will result where temporary surface ponding is used.

Summary

The facilities required for reasonably adequate drainage of an airfield vary greatly and cannot be defined precisely. Within the range of economic feasi-

(Using Curves No. 2.2 for Paved Areas and No. 1.6 for Turfed Areas)

Rate of Inflow into Underground Drains,[b] in Cubic Feet Per Second, Corresponding to the Value of t_c Adopted (Col. 10) for Inlet Numbers:																Inlet, or junction No.
16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	Total	
(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(1)
Runoff																
9.6	...	...	...	...	...	...	...	...	...	...	...	...	...	...	9.6	16
9.6	6.9	...	...	...	...	...	...	...	...	...	...	...	...	...	16.5	15
9.6	6.9	4.7	6.9	...	...	...	...	...	...	...	...	...	...	...	28.1	13
9.6	6.9	4.7	6.9	5.5	3.0	1.5	4.6	...	...	...	...	...	...	...	42.7	9
9.6	6.9	4.7	6.9	5.5	3.0	1.5	4.6	2.1	...	...	...	...	...	...	44.8	8
9.6	6.9	4.7	6.9	5.5	3.0	1.5	4.6	2.1	1.7	...	...	...	...	...	46.5	7
9.6	6.9	4.7	6.9	5.5	3.0	1.5	4.6	2.1	1.7	1.8	2.4	1.6	2.6	...	54.9	3
9.6	6.9	4.7	6.9	5.5	3.0	1.5	4.6	2.1	1.7	1.8	2.4	1.6	2.6	2.1	57.0	2
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	1
...	...	4.7	...	...	...	...	...	...	...	...	...	...	...	...	4.7	14
...	...	...	...	5.5	...	...	...	...	...	...	...	...	...	...	5.5	12
...	...	...	...	...	3.0	...	...	...	...	...	...	...	...	...	3.0	11
...	...	...	...	5.5	3.0	1.5	...	...	...	...	...	...	...	...	10.0	10
...	...	...	...	...	...	...	...	...	...	1.8	...	...	...	...	1.8	6
...	...	...	...	...	...	...	...	...	...	...	2.4	...	...	...	2.4	5
...	...	...	...	...	...	...	...	...	...	1.8	2.4	1.6	...	...	5.8	4
onding of Runoff																
22.0	...	...	...	...	...	...	...	...	...	...	...	...	...	...	22.0	16
22.0	20.2	...	...	...	...	...	...	...	...	...	...	...	...	...	42.2	15
22.0	19.6	21.5	19.6	...	...	...	...	...	...	...	...	...	...	...	82.7	13
22.0	19.6	21.5	19.6	15.9	15.1	6.9	17.7	...	...	...	...	...	...	...	138.3	9
20.7	18.9	20.2	18.9	15.3	14.6	6.6	17.2	9.9	...	...	...	...	...	...	142.3	8
20.7	18.9	20.2	18.9	15.3	14.6	6.6	17.2	9.9	7.5	...	...	...	...	...	149.8	7
20.0	18.3	19.5	18.3	14.2	13.6	6.4	16.0	9.5	7.2	7.4	10.5	6.7	11.8	...	179.4	3
20.0	18.3	19.5	18.3	14.2	13.6	6.4	16.0	9.5	7.2	7.4	10.5	6.7	11.8	10.0	189.4	2
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	1
...	...	21.5	...	...	...	...	...	...	...	...	...	...	...	...	21.5	14
...	...	...	...	15.3	...	...	...	...	...	...	...	...	...	...	15.3	12
...	...	...	...	...	15.6	...	...	...	...	...	...	...	...	...	15.6	11
...	...	...	...	15.9	15.1	6.9	...	...	...	...	...	...	...	...	37.9	10
...	...	...	...	...	...	...	...	...	...	8.0	...	...	...	...	8.0	6
...	...	...	...	...	...	...	...	...	...	...	11.3	...	...	...	11.3	5
...	...	...	...	...	...	...	...	...	...	8.3	11.3	7.5	...	...	27.1	4

...ues used in Cols. 11 to 25, Table 5(*a*); and Table 4(*b*), the discharge values used in Cols. 11 to 25, Table 5(*b*).

bility, the design capacity of the drainage systems should be adequate to remove surface runoff from the selected design storm without damage to field facilities, undue saturation of subsoil, or significant interruption of normal

traffic. Careful consideration should be given to drainage in the preparation of grading plans for an airfield. Results of studies indicate that the standard rainfall intensity-duration curves adopted as general standards provide a satisfactory degree of accuracy for airfield drainage problems.

The overland flow equations developed by Mr. Horton provide a useful tool for the determination of the rates of runoff from paved or turfed areas that would result from continuous and uniform rates of rainfall-excess. The procedures presented herein for the design of drainage facilities for airfields utilizing surface pondage for temporary storage of runoff from high intensity storms of short duration will provide a balanced and economical design consistent with the selected design storm rainfall. The experience to be gained in the operation of the drainage systems of airfields now completed should furnish data for the improvement of present procedures.

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DISCUSSIONS

ORGANIZING AND FINANCING SEWAGE TREATMENT PROJECTS

Discussion

BY SAMUEL A. GREELEY

SAMUEL A. GREELEY,[9] M. AM. SOC. C. E.[9a]—The discussions of this paper have covered a considerable range. That submitted by Mr. Adams is a good record of a practical experience in the accomplishment of sewage treatment on a regional basis by separate organizations. Experiences of this kind are useful in developing better methods of organizing and financing such projects.

Mr. Smith describes the procedure followed in determining the method of financing the Detroit (Mich.) sewage disposal project. He calls attention to one matter of importance and that is the approximation of a two-part rate in which one part is designated as a charge for usage based on metered water consumption and a second part is based on the financing of general obligation bonds by a property charge or tax. Sometimes this division approaches an equitable distribution of the cost and should not be overlooked.

Mr. Moore describes the successful organization of a small sanitary district. There are undoubtedly localities where a small sanitary district meets the requirements; but it seems likely that, in general, the duplication of governmental units for small populations is not desirable.

Mr. Kingsbury gives a clear illustration of the effect of the type of bond and method of borrowing on the annual cost illustrating, for instance, the difference between an equal total annual cost and an equal per capita annual cost. Fixed charges of this kind should be related to the annual sums set up for depreciation and the method of allocating major repairs or replacements as between capital and operating accounts. Consideration should also be given to the lower total expenditure in dollars with serial bonds as compared to sinking fund methods. Presumably the serial bond maturities and interest rates can be adjusted to approximate equal annual total amounts. Frequently, however, general con-

NOTE.—This paper by Samuel A. Greeley, M. Am. Soc. C. E., was published in December, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1942, by Messrs. Milton P. Adams, and Hal F. Smith; April, 1943, by Herbert Moore; May, 1943, by Francis H. Kingsbury; and September, 1943, by A. Brinton Carson.

[9] (Greeley & Hansen), Chicago, Ill.

[9a] Received by the Secretary October 20, 1943.

stitutional or charter provisions limit the choice, and the type of borrowing cannot be arranged to meet favorably different local conditions.

The discussion by Lieutenant Carson is of marked interest and describes a procedure for an ideal design in organizing and financing a sewage treatment project. The first seven sections under "Organization" relate primarily to the determination of the extent, area, and need of a sanitary district. In this connection, attention should be called to Chapter 112, Laws, Session of 1910 of the State of New Jersey. Paragraph 2 of this chapter authorizes governing bodies of any two or more municipalities in the state to unite and jointly cause to be made at their joint expense, by competent engineers, surveys, maps, plans, reports, and estimates of proposed works. This is a good method and should be used more widely. In the writer's opinion, the comments by Lieutenant Carson on methods of financing are somewhat limited, as full consideration is not given to the difference between systems handling sanitary sewage only and those with capacity for storm sewage as well. His description and outline of an ideal procedure, however, is very stimulating.

It would be highly desirable if experienced engineers were employed to advise in the preparation of enabling acts and in the formation of sanitary districts.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DETERMINATION OF KUTTER'S *n* FOR SEWERS PARTLY FILLED

Discussion

BY C. FRANK JOHNSON

C. FRANK JOHNSON,[21] M. AM. SOC. C. E.[21a]—Discussers have added some information on the variation of Kutter's *n* for conduits partly filled. This information relates largely to conduits carrying clean water, rather than to sewers; and to laboratory experiments, rather than to field measurements. As previously stated by the writer, such information affords a valuable guide for use in sewerage computations, but cannot be used without modification.

Professor Camp expresses regret that "the opinion is so widely held that measurements made in pipes carrying clean water are not applicable to the flow of sewage. Such is not the case for clean sewers." The writer is of the opinion that there is no such thing as a "clean sewer," even though there are sewers which are free of deposits and sediment. The nature of sewage is such that the sewer walls are inevitably affected over a period of years after the sewer is put in use, the walls acquiring a surface different from that found in a conduit carrying clean water. This surface condition, rather than the suspended matter in the sewage, is what affects the flow in a sewer. The flow pattern in a sewer would probably be substantially the same for experiments made with clean water flowing in the sewer as for those made with sewage. Professor Camp's study of the ratios of friction factors for various depth-diameter ratios is of considerable interest.

Mr. Coulter mistakenly attributes to the writer the conclusion that higher values of *n* at low flows may result from friction between the sewage surface and the air. This conclusion was merely quoted from another published article. Mr. Coulter's explanation of the variation of *n* is of interest, and the writer has no quarrel with it. However, attention is again directed to the fact that the

NOTE.—This paper by C. Frank Johnson, Assoc. M. Am. Soc. C. E., was published in February, 1943, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1943, by Messrs. Thomas R. Camp, Richard G. Coulter; and September, 1943, by C. E. Ramser.

[21] Chf. Engr., Commrs. of Sewerage of Louisville, Louisville, Ky.

[21a] Received by the Secretary December 17, 1943.

flow in sewers is affected to some degree by the peculiar condition of the walls, as well as by factors affecting other conduits.

The data contributed by Mr. Ramser are most valuable, ranging as they do from a condition of the conduit being about one fourth filled to completely filled.

In conclusion, the writer urges that additional field measurements be made in sewers partly filled, to supplement the meager information on this subject. Reliance on laboratory and other measurements made in conduits carrying clean water is not entirely satisfactory for sewerage work.

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DISCUSSIONS

CHARACTERISTICS OF HEAVY RAINFALL IN NEW MEXICO AND ARIZONA

Discussion

BY WALTER B. LANGBEIN, AND EMIL P. SCHULEEN

WALTER B. LANGBEIN,[24] ASSOC. M. AM. SOC. C. E.[24a]—In general, rainfall regimes for engineering purposes are completely defined only by evaluation of the depth-area-intensity-frequency characteristics—a four-dimensional problem that has nowhere been solved. Merrill Bernard[25] and Gail A. Hathaway,[26] Members, Am. Soc. C. E., have described the latest methods of deriving depth-area-intensity relations, frequency being omitted. Mr. Yarnell's useful publication[2] treats the depth-intensity-frequency relations of "point" rainfall at first-order Weather Bureau stations exclusively, omitting area of storm rainfall. In the western part of the United States the sparsity of first-order stations imposes a serious shortcoming. The author contributes much toward filling this need by incorporating information from all stations, although his compilation is limited in turn because of this broader coverage to 24-hr periods. Thus, he reveals an expectancy of 24-hr rainfall of 6 in. or more once in 50 years on the average in Arizona, whereas Mr. Yarnell showed nothing more than 3.5 in. The deviation from Mr. Yarnell's results in New Mexico, although appreciable, is not so marked.

Isopluvial maps such as Fig. 5 are open to many kinds of errors, including errors of measurement, errors of sampling (the next 30 years might produce materially different frequency curves), and also errors arising from necessary generalization. Such errors are inherent in the statistical methods available to the engineer and are not peculiar to Mr. Leopold's expert handling.

NOTE.—This paper by Luna B. Leopold, Jun. Am. Soc. C. E., was published in February, 1943, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1943, by Lawrence Pratt, Assoc. M. Am. Soc. C. E.; June, 1943, by Messrs. L. L. Harrold and A. J. Dickson, and James Girand; November, 1943, by Clarence S. Jarvis; and December, 1943, by Paul V. Hodges, Edgar E. Foster, R. W. Davenport, and A. K. Showalter.

[24] Associate Engr., U. S. Geological Survey, Washington, D. C.

[24a] Received by the Secretary November 16, 1943.

[25] "Primary Rôle of Meteorology in Flood Flow Estimating," by Merrill Bernard, *Proceedings*, Am. Soc. C. E., January, 1943, p. 105.

[26] *Ibid.*, October, 1943, p. 1310.

[2] "Rainfall Intensity—Frequency Data," by David L. Yarnell, *Miscellaneous Publication No. 204*, U.S.D.A., August, 1935.

Statistical theory is not at all clear even in the matter of fitting a frequency time scale to the annual data. Mr. Leopold has computed his plotting positions by the California method, in which recurrence-intervals are computed by the formula $\frac{y}{m}$ (y being the number of years in the series, and m, the relative rank of an annual rainfall, listed in decreasing order of magnitude). Equally legitimate is the method of listing in the reverse order, in which case the interval in years is $\frac{y}{y-n}$ (n being the relative rank beginning with the lowest as 1). The values obtained by the second formula are generally called exceedance intervals. A graph plotted to conform with the recurrence-interval plotting positions will be higher than the exceedance-interval graph (that is, for a given interval in years the former will give the higher rainfall). There are other methods of plotting[27,28]; all produce graphs that lie between the recurrence-interval graph and the exceedance-interval graph. A common method is the Hazen compromise, $\frac{y}{m-0.5}$ or $\frac{y}{y-n+0.5}$, these two ratios becoming numerically equal. However, it is desirable to emphasize that Mr. Leopold's results, particularly the extrapolated 50-yr and 100-yr rainfalls, are higher than would be obtained by any other method of plotting now in use, applying the same data.

Assuming that the frequency data in Table 3 and Fig. 5 are accepted, this paper presents a wealth of material for study and analysis. For example, a search might profitably be made to find the effects of altitude or other measurable factors on rainfall intensity. The following statement by G. N. Brancato[29] seems pertinent to such an inquiry:

> "However, contrary to published and popular accounts, the thunderstorms produced the greatest amount of precipitation at the lower elevations and not on the mountain slopes. The most favorable condition for the production of heavy rain is the presence of an air mass with a sufficient amount of available energy and the greatest possible amount of moisture. Orographic lifting is very effective as a mechanism to release the latent energy in an air mass but as the air is lifted over progressively higher terrain the total amount of available precipitable water above any given area becomes progressively smaller. Over the lower level areas therefore there is always more potential rain waiting to be released, but the opportunity for realization occurs less frequently than at the higher elevations which are favored by orographic lifting."

Fig. 9 shows results of averaging groups of stations of like altitude for the summer in Arizona. The graphs show that, on the average, rainfall intensities are greatest in the range of altitudes between 3,000 ft and 4,000 ft. The decline in expected intensities at high altitudes is especially noteworthy. Furthermore, the spread between the 5-yr and 50-yr recurrence-interval graphs

[27] "Statistical Analysis in Hydrology," by L. R. Beard, *Transactions*, Am. Soc. C. E., Vol. 108 (1943), p. 1110.

[28] "On Serial Numbers," by E. J. Gumbel, *Annals of Mathematical Statistics*, Vol. 14, No. 2, June, 1943, pp. 163–178.

[29] "The Meteorological Behavior and Characteristics of Thunderstorms," by G. N. Brancato, Hydrometeorologic Section, U. S. Weather Bureau, April, 1942 (processed).

is greater at low altitudes than at high altitudes. Indeed, the indicated 50-yr rainfall is greater on the average at altitudes below 1,000 ft than at the 7,000-ft level. However, 5-yr rainfall is greater at the 7,000-ft level; and 10-yr rainfall is nearly the same at both levels. These relationships probably result from greater thunderstorm frequency at high levels due to instability created by lifting of the air. Seemingly, when, at rare intervals, conditions favor the development of critical storms at low levels, the resulting intensities are apt to be greater because of the greater amount of precipitable moisture in the air column.

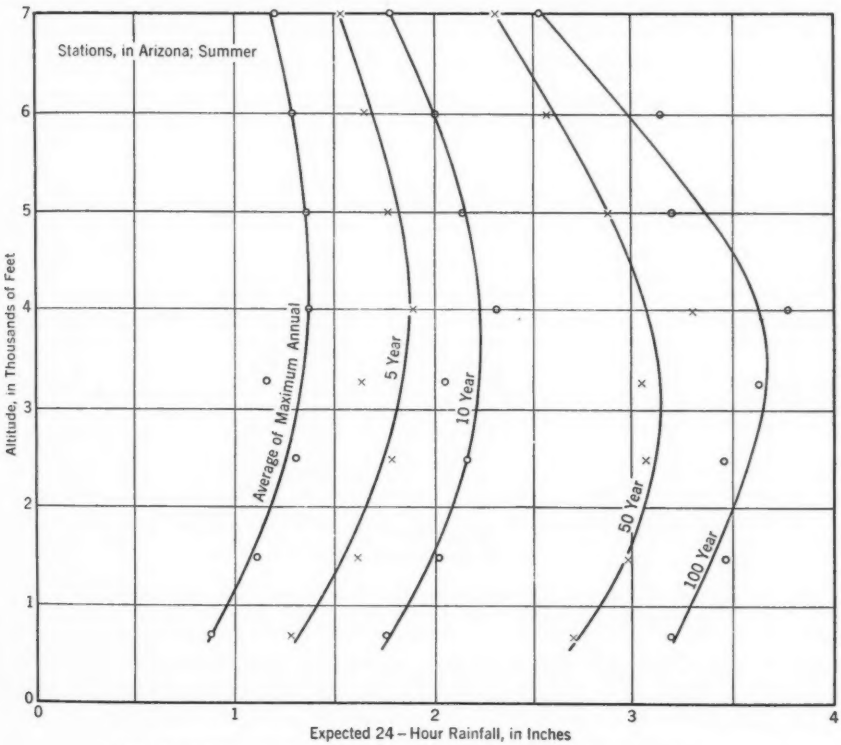

FIG. 9.—STUDY OF RELATION BETWEEN ALTITUDE AND EXPECTED 24-HR RAINFALL OF INDICATED RECURRENCE INTERVALS

These ideas are further explored in Fig. 10. There is a 20% difference between the "summer" and the "all-year" curve at altitudes higher than 6,500 ft (see Fig. 10(*a*)), indicating that non-summer rainfalls are important contributors to storm intensities at high altitudes. At stations below 1,100 ft (Fig. 10(*b*)) the spread is much less, especially at the upper end. At low altitudes in Arizona, the characteristics of rainfall intensities are seemingly dominated by those of summer. The difference in slope of the high-altitude and low-altitude graphs indicates a greater range in expected intensities at low altitudes than at high altitudes, as already has been mentioned.

Mr. Brancato outlines two influences on the intensity of rainfall produced by thunderstorms: (1) Lifting forces, and (2) amount of precipitable moisture. Altitude is a measure of the lifting indigenous to a station. It would seem that, for stations at the same altitude, mean seasonal or annual precipitation might furnish an approximate measure of such meteorologic and local effects as are indicative of precipitable moisture. Furthermore, comparisons of intensities at stations having the same mean seasonal or annual rainfall would provide a basis for study of the effect of altitude separately. These studies are made most simply by means of a multiple-correlation chart.

Table 3 does not list mean summer precipitation, and therefore "all-year" data have been used in this correlation. Altitude and mean annual precipitation have been correlated with "average of maximum 1-day rainfall" for "all

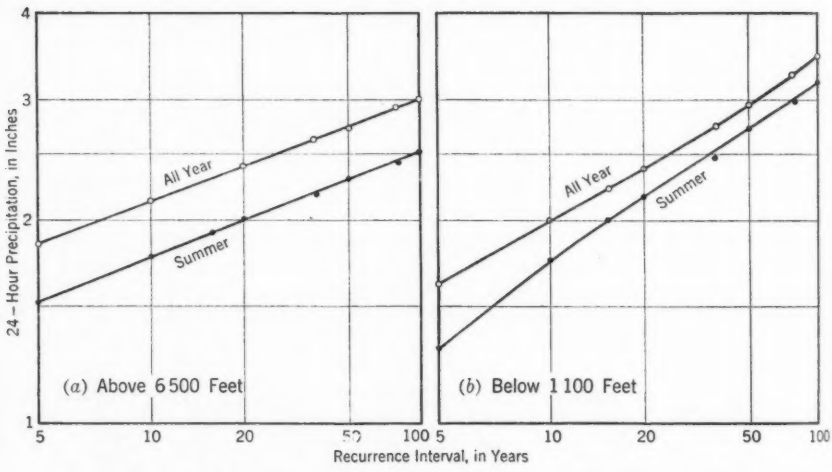

FIG. 10.—COMPARISON OF AVERAGE FREQUENCY CURVES FOR SUMMER AND ALL-YEAR, AT ALTITUDES BELOW 1,100 FT AND ABOVE 6,500 FT, IN ARIZONA

year." Separate correlations would need to be made for each recurrence-interval as well. For the longer recurrence-intervals the correlations become less accurate.

Diagrams were prepared on which were plotted altitude as ordinate and "average maximum 24-hr rainfall" as abscissa; and mean annual precipitation was written along each plotted point. On this background of points, contours of equal mean annual precipitation were drawn. The resultant contours are shown in Fig. 11 for Arizona and New Mexico, respectively. The mean error of estimate in Fig. 11(*a*) is 0.15 in., and in Fig. 11(*b*), 0.20 in. The decrease in rainfall intensity with altitude, mean annual precipitation constant, is to be noted. Also, for a given altitude there is a marked increase in rainfall intensity with increase in mean annual precipitation. Diagrams similar to Fig. 11 should provide means for estimating rainfall intensity at places in Arizona and New Mexico when the altitude and mean annual precipitation are known.

Mean annual precipitation can be interpolated from isohyetal maps modified if necessary on evidence given by native vegetation.

Any generalization, whether by graph or map, however, would be subject to erratic variation because of the great divergence in relief in the mountainous

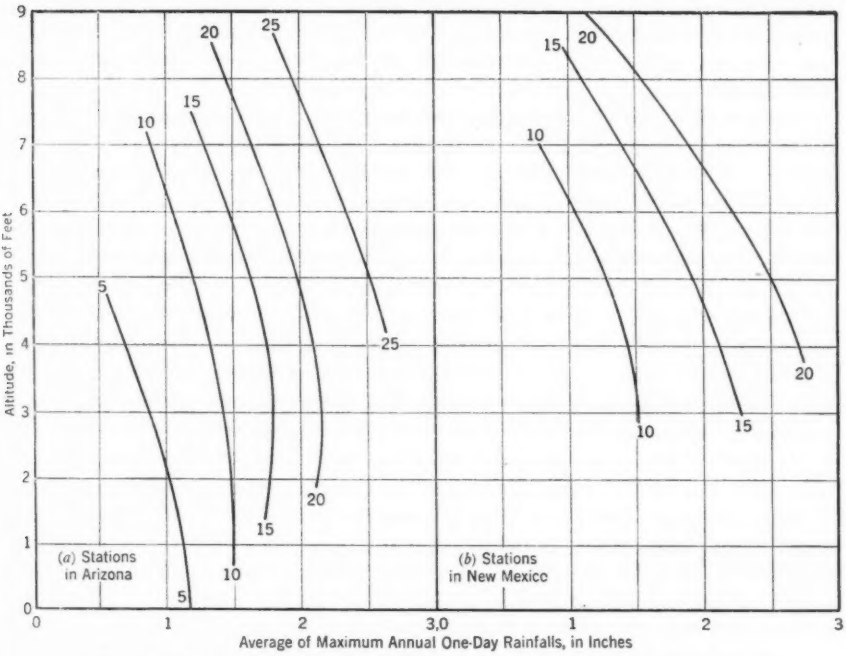

FIG. 11.—RELATION OF ALTITUDE AND AVERAGE OF MAXIMUM ANNUAL 1-DAY RAINFALLS IN TERMS OF MEAN ANNUAL PRECIPITATION

and intermountainous parts of the West. Measurements of rainfall provide the best and only reliable guide to storm frequency and intensity. As Mr. Leopold states, caution is imperative in transferring the results to locations distant from the place of observation, especially if the topographic setting or orientation with respect to the moisture-carrying winds is markedly different.

EMIL P. SCHULEEN,[30] ASSOC. M. AM. SOC. C. E.[30a]—In making available to the engineering profession the results of his extensive study, the author is to be commended. A great amount of the detailed work involved in his analyses apparently was done by a number of workers from the Work Projects Administration. The author was fortunate in having had such assistance at his disposal. The writer wonders, however, if (as is sometimes the case when assistance is available in abundance) the work has not been reduced largely to mechanical procedures with efforts directed toward volume rather than reliability. In this respect he has reference particularly to the extensive

[30] Senior Engr., U. S. Engr. Office, Pittsburgh, Pa.

[30a] Received by the Secretary November 26, 1943.

results summarized in Table 3, which will be discussed further in later paragraphs.

The section on "Seasonal Distribution of High Rainfalls" might well have followed immediately after the section on "Storm Types," for these two sections, together with Fig. 2, provide an excellent treatment of the relation of storm types and topographic relief to the intensity, geographic distribution, and seasonal occurrence of rainfall in the various parts of the area under consideration.

In the last paragraph under the heading, "Aerial Pattern of Summer Storms," it is implied that the depth-area data given in Table 1 are but a few samples, and that other similar data are available, although considered inadequate to indicate depth-area-time patterns for design purposes because of the lack of definite information concerning intensities. Apparently the limitation on the use of the available data for making such analyses applies primarily to the case of small areas where rainfalls of a few hours duration would govern design requirements. Would not the available data, however, permit analyses for larger areas where rainfalls of 1-day or 24-hr duration would be important? Data of that nature, separated in accordance with the summer-type and winter-type storms, would provide a valuable addition to the author's paper in the interest of design storms and floods for larger drainage areas.

In the first paragraph under the heading, "Rainfall Intensity," it is indicated that the mass curves in Fig. 1(*a*) represent the most intense rainfalls on record at recording gage stations in New Mexico and Arizona. In the second paragraph of this section it is indicated that the points in Fig. 1(*b*) were obtained from the records of standard non-recording rain gages where the times of beginning and ending of rainfall had been noted by the observer. The writer is not familiar with the available data, but suggests that it may have been possible to develop reasonably accurate mass curves for some or many of the 72 storms represented, by comparing the records from the non-recording stations with those from recording stations in the vicinity and by comparing the times of beginning and ending of rainfall at the respective stations. This analysis would be facilitated further if the regular time of observation differed materially among the several stations, as is frequently the case. If such mass curves were available, presumably their envelope would approximate the envelope curve shown in Fig. 1(*b*). This envelope curve, therefore, would serve as the mass curve for the design storm, if the project were to be designed for the maximum storm possibility.

The author states that the frequency data given in Table 3 are based on records for 15 years or more from standard non-recording rain gages, and that in a few instances shorter record periods were used. There is reason to question the reliability of frequency determinations for intervals of as many as 100 years when the studies are based on records for as few as 15 years. Most extensive rainfall records indicate tendencies for cyclic variation with alternate periods of heavy and light rainfall. Frequency studies based on short periods of record, therefore, may not reflect truly average conditions. There is also the possibility in the use of short records for one or two abnormal or subnormal values

to warp the true tendency of the remaining data. The author has recognized this possibility by his reference (see heading, "Frequency of High Rainfalls") to "aberrant points in the upper limits of the series." It is noted also that in some cases the rainfall values were "24-hr values" based on the observer's notations with respect to the time of beginning and ending of rainfall, which in some instances indicated a total lapsed time of considerably less than 24 hr, whereas in other cases the values were "1-day values" as obtained from consecutive regular daily observations. It is noted further that, where rains continued through two or more days, the largest amount recorded for one day was used. All these variations in the manner of applying the basic data, while difficult to avoid, affect the reliability of the results.

Illustrations of apparent inconsistencies in results may be had from a study of the rainfall values for various frequencies given in Table 3, in conjunction with reference to the relief maps of Figs. 3 and 4. The writer has selected two instances as examples. The first of these involves a comparison of the stations at Tatum and Prairieview, N. Mex. These two stations differ but 125 ft in elevation and are only 10 or 12 miles apart in an area where the topographic relief apparently is uniform and similar, yet the 50-yr value for the first is almost twice that for the second. The second instance involves a comparison of the stations at Palo Verde, Roy, Solano, Hoosier Ranch, and Mosquero, in New Mexico. These stations range from El. 5579 to El. 5884, an over-all difference of 305 ft. All five stations are located within a radius of 6 or 7 miles in an area which, from Fig. 4, appears to be relatively uniform and similar in topographic relief; yet the 50-yr rainfall values range from 3.77 to 6.80, an over-all difference of about 80%. Two of these stations, Palo Verde and Roy, are indicated by Fig. 4 to be only 3 or 4 miles apart and by Table 3 to differ in elevation by only 4 ft, yet the 50-yr rainfall value for the second is more than 30% greater than that for the first. One might expect that rainfall frequencies should be similar at the various stations in these respective areas, unless there are topographic features, not apparent in Fig. 4, that are of sufficient extent to explain the differences in the rainfall frequency values given for those stations in Table 3. If no such explanation can be found, these differences must be attributed to inconsistencies in the basic data or to the method in which the data have been applied to obtain the results.

In commenting on the fact that Table 3 indicates certain stations to have the same 24-hr rainfall expectancy at a given recurrence-interval for summer as for the entire year, the author makes the statement that (see heading, "Frequency of High Rainfalls") "This simply indicates that that station characteristically received the largest 24-hr rainfalls in the summer period and, therefore, the recurrence-interval curves for summer and all year would be practically identical." It is recognized that this might be true in some cases, but in others it is difficult to understand why it should be true with respect to one station and not with respect to another in the immediate vicinity and in the same general topographic setting. Referring again to the stations at Tatum and Prairieview, Table 3 indicates the 50-yr values for the former to be 7.70 and 8.00 for summer and all year, respectively, whereas in the case of the latter both values are given as 4.08. Unless (as mentioned in the first com-

parison of these two stations) there are topographic features, not apparent in the relief map, that affect one station and not the other, it does not appear reasonable that the greatest rainfalls characteristically should occur in the summer at one station and during the remaining seasons at the other.

Under the heading, "Isopluvial Maps," it is stated that "The small areal extent of high rainfalls occurring in summer-type storms is a factor leading to differences between stations since any high-intensity, individual storm rarely is recorded at more than one station." By this statement the author indicates that the records in some instances are erratic. It is probable also that, in many instances, the greatest rainfall during a high-intensity storm was not recorded at any standard station. Where such spotty rainfall occurs in a region of relatively uniform topographic relief, and heavy rainfall is recorded at one station and not at another near-by station, the location of the point of heaviest rainfall must be attributed to mere chance. Presumably, if both stations had long record periods, the occurrence of high rainfall values would be about the same at each and their frequency curves would approach one another.

The writer's comments, particularly with respect to the variable nature of the results in Table 3, are made with the intent of calling attention to possible sources of inconsistencies in the basic data and in their application leading to the tabulated results. He is not familiar with the topographic details of New Mexico and Arizona and his comments regarding the effects of such details on rainfall intensity and distribution are based solely on the apparent variations in relief indicated by Figs. 3 and 4.

The paper as a whole is very well prepared and presents much valuable information to the engineering profession. It is suggested, however, that the value would be enhanced if the results in Table 3 were classified as "excellent," "good," "fair," etc., in accordance with the author's knowledge of the relative merits of the basic data and his results.

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DISCUSSIONS

EFFECT OF TURBULENCE ON SEDIMENTATION

Discussion

BY E. R. VAN DRIEST

E. R. VAN DRIEST,[11] ASSOC. M. AM. SOC. C. E.[11a]—An excellent example of the mathematical analysis of an engineering problem is presented in this paper. It clearly demonstrates the procedure followed in such a treatment; namely, the establishment of the basic assumptions, the derivation of the fundamental differential equation describing the physical phenomenon involved, the application of the boundary conditions, and the finding of a solution of the differential equation that satisfies the boundary conditions. Furthermore, the paper presents experimental data to substantiate the basic assumptions and the assigned boundary conditions.

The writer is not in full accord with the derivation of Eq. 17*b*. The assumption that $\epsilon_x \frac{\partial^2 c}{\partial x^2} = 0$ does not mean that $\frac{\partial c}{\partial x} = 0$, or that there is no diffusion of suspended matter in the x-direction. However, if it is assumed that the mean concentration is constant all along the x-axis, then there will be no diffusion in that direction; whereupon both $\frac{\partial^2 c}{\partial x^2}$ and $\frac{\partial c}{\partial x}$ will be zero and Eq. 17*b* will become, for the required steady state,

$$\epsilon \frac{\partial^2 c}{\partial y^2} + w \frac{\partial c}{\partial y} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (74)$$

which of course leads back to the basic Eq. 3. Eq. 17*b* is a case of steady state, but this necessitates a diffusion in the x-direction. Furthermore, the statement concerning the mathematical identity of Eq. 17*b* and Eq. 17*c* is rather difficult to follow. Certainly the solutions of these equations are independent of one another since c in the one case is a function of x and y whereas c in the other is a function of y and t. However, the statement could be interpreted so that the effect of the right-hand member of Eq. 17*b* in maintaining steady

NOTE.—This paper by William E. Dobbins, Jun. Am. Soc. C. E., was published in February, 1943, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1943, by A. M. Gaudin; May, 1943, by John S. McNown; June, 1943, by Messrs. H. A. Einstein, and Thomas R. Camp; and October, 1943, by Messrs. Garbis H. Keulegan, C. W. Thornthwaite, and A. A. Kalinske.

[11] Associate Prof., Dept. of Civ. Eng., Univ. of Connecticut, Storrs, Conn.

[11a] Received by the Secretary November 1, 1943.

conditions is the same as the effect of the right-hand member of Eq. 17*c* in causing growth.

As a matter of generality, it can readily be shown, upon consideration of the flux of sediment into and out of a small element of volume, that the general three-dimensional equation for diffusion in a liquid stream is given by

$$\frac{\partial c}{\partial t} + U\frac{\partial c}{\partial x} + V\frac{\partial c}{\partial y} + W\frac{\partial c}{\partial z}$$

$$= \xi\frac{\partial c}{\partial x} + \eta\frac{\partial c}{\partial y} + \zeta\frac{\partial c}{\partial z} + \frac{\partial}{\partial x}\left(\epsilon_x\frac{\partial c}{\partial x}\right) + \frac{\partial}{\partial y}\left(\epsilon_y\frac{\partial c}{\partial y}\right) + \frac{\partial}{\partial z}\left(\epsilon_z\frac{\partial c}{\partial z}\right) \quad \ldots(75)$$

in which U, V, and W are the stream velocities in the x-direction, y-direction, and z-direction and ξ, η, and ζ are the settling velocity components in the directions of $-x$, $-y$, and $-z$, respectively. Hence, it is seen that, for horizontal flow parallel to the x-axis with uniform turbulence and sediment concentration a function of y and t only, the y-axis being vertical—

$$\frac{\partial c}{\partial t} = \epsilon\frac{\partial^2 c}{\partial y^2} + \eta\frac{\partial c}{\partial y} \quad \ldots(76)$$

Eq. 76 is the same as Eq. 17*c* when $\bar{w}$ is substituted for η. It will be noted that in the one-dimensional case the concentration is independent of the stream velocity. The fact that Eq. 75 becomes the general heat conduction equation when ξ, η, and ζ are placed equal to zero is interesting.

The author has very cleverly simplified the boundary condition at the bottom of the cylinder by requiring constant hydraulic conditions to prevail at the bed. That the pickup, in the case of run V, for such conditions is constant with time for small concentrations near the bed is probably true as can be inferred from the physical picture, but the pickup would undoubtedly change for larger concentrations near the bed because of the effect of the sediment on the turbulence mixing and the mutual interference of the solid particles. It seems that this action should be somewhat analogous to that of the transfer of heat from a wall of fixed temperature to a turbulent fluid; here the eddies coming in contact with the wall carry away heat at a rate depending upon the mean temperature of the eddies near the wall. The fact that the experimental data agree closely with the theoretical curves proves the author's choice of this bottom condition to be valid.

The author should be commended for his perseverance and ingenuity in accomplishing the theoretical analysis and performing the experimental work.

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DISCUSSIONS

DEVELOPMENT OF THE CHICAGO TYPE BASCULE BRIDGE

Discussion

BY C. B. MCCULLOUGH

C. B. MCCULLOUGH,[10] M. AM. SOC. C. E.[10a]—The development of the "Chicago type" bascule structure is an interesting and clear-cut illustration of the adaptation of design types to meet special or particular local conditions. Numerous advantages inhere in this type, principal among which may be mentioned:

(1) Exterior blocking of the channel is avoided during operation. This advantage over the swing bridge assumes considerable importance in localities where it is essential to maintain maximum docking facilities along the river close to the structure, especially in those relatively narrow waterways such as the Chicago River, where the movable span constitutes the greater part of the stream crossing.

(2) It is possible for river traffic to approach within a comparatively short distance of a bascule structure, whereas for swing spans, craft must stand off much farther on account of the difficulty in negotiating a split channel and swinging around the draw rest.

(3) A bascule may be raised slightly to permit the passage of small boats which fail to clear by only a small margin, whereas, in general, a swing span must be opened a full 90° for each vessel regardless of its vertical clearance. Because of this fact, bascule spans may be operated with much less interference to vehicular traffic, particularly in those instances wherein a great portion of the river traffic is relatively low-masted.

(4) For the bascule it is not necessary, as in the case of the swing span, to construct a large pivot pier in the center of the channel. In certain locations, such a pier structure operates to deflect shoreward currents, with consequent destructive erosion along adjacent banks.

NOTE.—This paper by Donald N. Becker, M. Am. Soc. C. E., was published in February, 1943, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1943, by Alonzo J. Hammond, Past-President and Hon. M. Am. Soc. C. E.; and September, 1943, by Messrs. Clinton D. Hanover, Jr., and Armour T. Granger.

[10] Asst. Chf. Engr., State Highway Dept., Salem, Ore.

[10a] Received by the Secretary November 26, 1943.

(5) The bascule span, unlike the swing span, does not require the construction of a draw rest in the center of the channel, which construction always operates as a serious obstruction to the channel and a menace to water-borne traffic.

(6) It is possible to construct wide roadway structures (such as are necessary for traffic conditions in cities the size of Chicago) without the necessity of blocking the channel. This condition does not always prevail in the case of swing spans, for the reason that there is a definite relationship between the width of the roadway and the minimum dimensions that may be adopted safely for the pivot pier. If the roadway width is increased beyond a certain limit, the pivot pier becomes of such dimension as to obstruct the channel seriously.

(7) The bascule type lends itself to widening with a minimum of inconvenience to traffic because of the possibility of building additional roadway lanes immediately adjacent and alongside the existing structure, whereas such procedure is impossible in the case of a swing span because of the necessity for the maintenance of swing clearance during construction.

(8) In the bascule structure (in certain types at least), the raised leaves may be made to constitute an effective traffic barrier. This is a tremendous advantage as compared to the swing span, particularly in the case of heavy, high-speed traffic arterials, whereon accident elimination and control pose serious and challenging problems.

(9) As compared with the swing span, in general, the bascule structure is safer from the standpoint of water-borne traffic in that it eliminates the possibility of collision with the open end of the span when it is swung.

Most of the foregoing advantages derive from a comparison of the bascule bridge with the ordinary swing span. However, some of them are also applicable when such type is compared with the vertical lift. In addition, it appears possible in the case of the bascule, particularly the double-leaf design, to develop a greater degree of esthetic or architectural excellence than in the case of either the swing span or the vertical lift.

The foregoing items constitute the principal advantages inherent in the bascule type, some of which were mentioned by Mr. Becker.

The paper covers the origin and development of one specialized form of bascule, which has justly been denominated the "Chicago type," The historical development which he traces is most interesting. Of special interest to engineers is his mention of the internal rack (heading, "The Improvement Period") developed by Mr. von Babo; the utilization of transverse trunnion girders; the gradual development of concrete as a counterweight material; the utilization in one instance of an operating rack attached to the side, instead of in the plane, of the truss; the utilization of roller bearings in lieu of phosphor-bronze bushings; and the use, in one instance, of hydraulic motors.

The Chicago type represents a particular adaptation to meet the demands imposed by local conditions. The entire movable ensemble rotates about one single trunnion. In comparison with the multiple-trunnion types, this development presents the advantage of simplicity in construction and at the same

time operates to reduce to a minimum the number of parts needing lubrication. As compared to the rolling lift type, the Chicago type concentrates the center of gravity at one fixed point during all phases of the operation. This type also discloses certain esthetic advantages as compared to most of the other types in common use. As mentioned by Mr. Becker, it is possible through the utilization of an arched lower chord, a full deck structure, and an appropriate architectural treatment of the operators' houses to produce an ensemble much more pleasing than is generally possible in other types. Fig. 21 is a view of a structure in this category which spans the Siuslaw River on the Oregon Coast Highway. It is illustrative of the possibilities in double-leaf simple trunnion bascule design in combination with reinforced concrete approach construction. The flanking approach spans of this bridge are 154-ft reinforced concrete bow string arches.

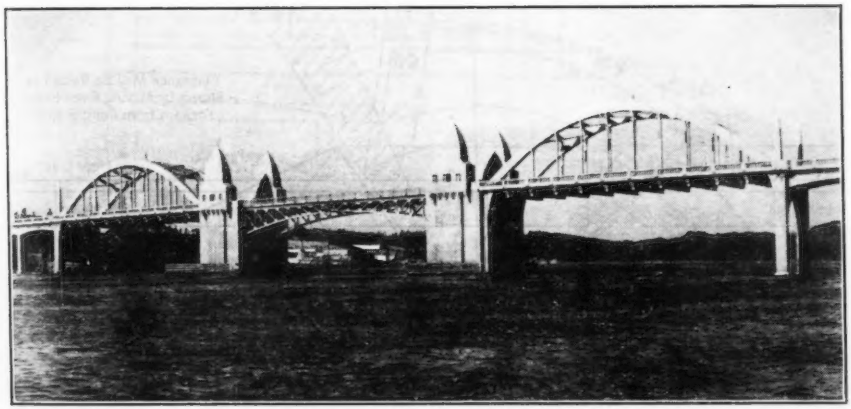

FIG. 21.—150-FT ELECTRICALLY OPERATED DOUBLE-LEAF BASCULE SPAN ACROSS SIUSLAW RIVER ON OREGON COAST HIGHWAY

Another advantage inherent in the Chicago type is the facility with which the raised leaf, in single-deck structures, can be converted into a traffic barrier. Fig. 22 is a line drawing of the shoreward end of a Chicago type span showing the location of the trunnion in reference to the break in the roadway floor. With a fixed total angle of opening, ϕ, the clearance C between the fully raised leaves and the fixed roadway deck may be varied simply by moving the roadway break fore or aft along the longitudinal axis of the deck; in other words, simply by varying the angle θ as indicated. In view of the difficulty in safeguarding the operation of modern high-speed highway traffic, a positive barrier of this kind presents an advantage that cannot be overestimated.

A most important feature in the design of bascule bridges is that of controlling the sequence of the various events that compose an operating cycle. In general, such a cycle comprises the following incidents, which should occur in the order given:

Opening Cycle

(1) Operation of warning lights, signs, or signals;
(2) Closing of remote roadway gates, if any;
(3) Closing of near roadway gates, traffic barriers, and the opening of derails, if any;
(4) Unlocking the leaves;
(5) Opening the span.

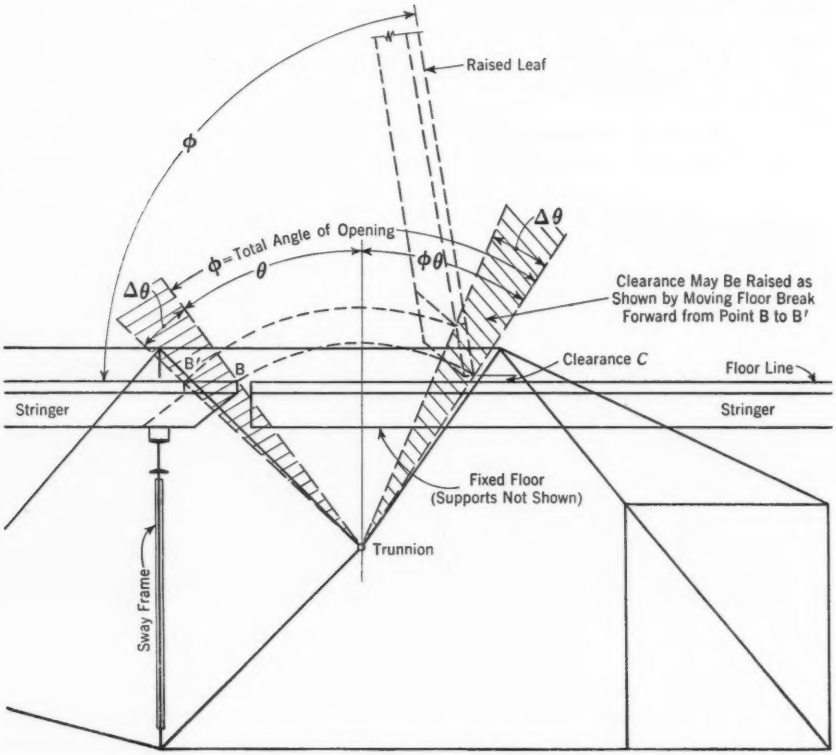

FIG. 22.—EFFECT OF VARYING OPENING IN DECK

Closing Cycle

(1) Closing the span;
(2) Locking the leaves;
(3) Raising the traffic barriers, near gates, and closing the derails if any;
(4) Raising the remote gates.

The warning signs or signals may consist of bells, sirens, or whistles for the purpose of an audible warning, and the use of lights and semaphores for visual warning. The bells or sirens may be operated simultaneously with the closing of the gates or they may cease before the gates are lowered. In general, a prolonged siren is objectionable in that it may drown out signals from river

craft, for which reason it is generally the part of wisdom to terminate the siren signals before the roadway gates are lowered. However, this objection does not hold in the case of a traffic bell or gong, which may be kept ringing during the entire operation of the roadway gates. Lights or semaphore arms may be interlocked electrically with the roadway gates in such manner that the gates cannot be operated until such lights or semaphore arms are set to "danger." For example, an ordinary door-type switch may be cut into the gate circuit and installed on the semaphore arm in such manner that the roadway gate motor will remain dead or inoperative until the semaphore is set in its warning position, as indicated in Fig. 23. In a similar manner the traffic-light circuit may be interlocked to operate a relay which closes the gate circuit when, and only

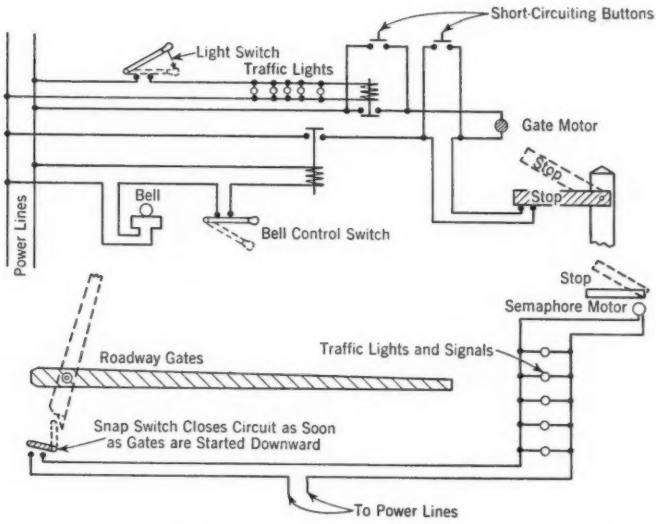

FIG. 23.—METHOD OF INTERLOCKING CONTROL OF GATES AND WARNING DEVICES

when, the light circuit is energized. It is also possible to arrange the circuits operating semaphore arms and traffic lights in such a way as to energize such circuits the instant the gate moves from its fully open position. Both of these arrangements and also a system of short-circuiting switches are indicated schematically in Fig. 23. In any interlocked operation of this kind, a short-circuiting arrangement is necessary; otherwise it is possible for electrical interlocking to interpose dangers as great as those which the ensemble is designed to avoid. The foregoing constitutes the essence of the first step in the electrical interlocking of operations.

If, in addition to the roadway gates indicated in Fig. 23, the installation comprehends a second system of gates (near gates), traffic barriers or derails, the interlocking obviously should be extended so that it is impossible to close these devices unless and until the remote gates are closed.

It is obviously necessary to lock the leaves of a double-leaf bascule bridge together in order that the structure may transfer live-load shear. This locking is usually accomplished by a center-locking arrangement, and, in addition, tail locks are sometimes used, as described by the author. Such devices are, in general, operated by means of a master switch or controller, and the wiring should be so arranged as to interlock with previous warning signals, gates, and barriers so that such locking devices cannot be released until all the signals, gates, barriers, etc., have been properly operated and set to warning. A schematic arrangement of this kind is indicated in Fig. 24.

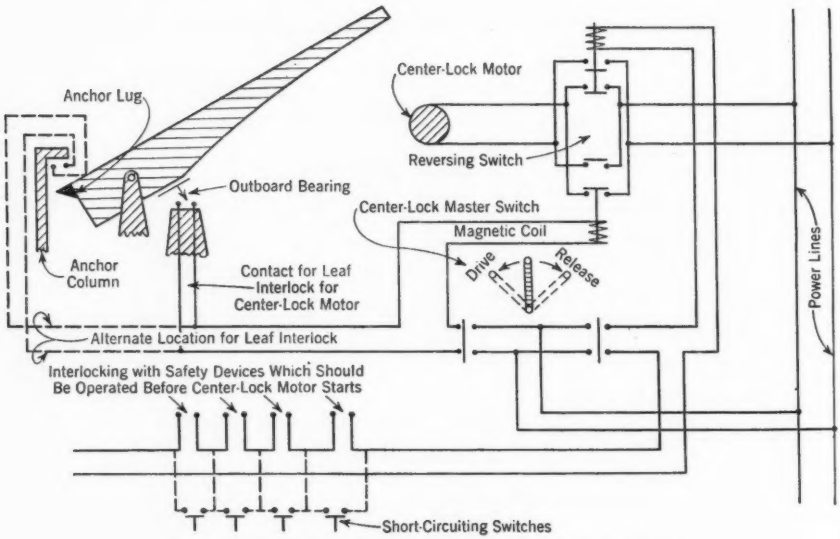

FIG. 24.—WIRING OF CONTROLS ON THE CENTER-LOCK MOTOR

In addition to the motors controlling the locking devices it is necessary, of course, to install motors for the purpose of operating the leaves. Such motors generally are controlled through a master switch. This switch should be wired through a contact on the center lock which is closed only when the center-lock pin is fully released, thus avoiding the possibility of damage by attempting, negligently, to raise the leaves before they are unlocked. A schematic arrangement of this kind is indicated in Fig. 25.

In closing, the leaves of a bascule structure are generally lowered to a seat on an outboard or live-load shoe at the stream edge of the abutment and anchored at a rear bearing, as indicated in Fig. 24, to preclude the possibility of live load on the trunnion. At either of these points it is possible to install a device to complete an otherwise open circuit for the center-lock or tail-lock motors in the "drive" direction, so that these devices will remain inoperative in that direction until the bridge leaves are down, fully seated and correctly alined. If this interlock is not provided, the center or tail locks can be driven while the bridge is in the open position. Although this in itself is not particu-

larly objectionable (since the main motor circuit is wired so as to be inoperative except for these devices fully open or drawn) there is some danger of jamming the center or tail-locking devices by attempting to drive them when the leaves are not fully seated and exactly alined, except in those cases where the locks are designed so as to draw the leaves together during the last part of their movement. When the wiring is such as to permit the operation of the center-locking pins with the leaves partly open, it appears that provision should be made for making the roadway gates, traffic barriers, etc., inoperative when the leaves are raised, since otherwise such gates or barriers could be raised with the leaves partly or fully open. On the other hand, if the gate and barrier circuits

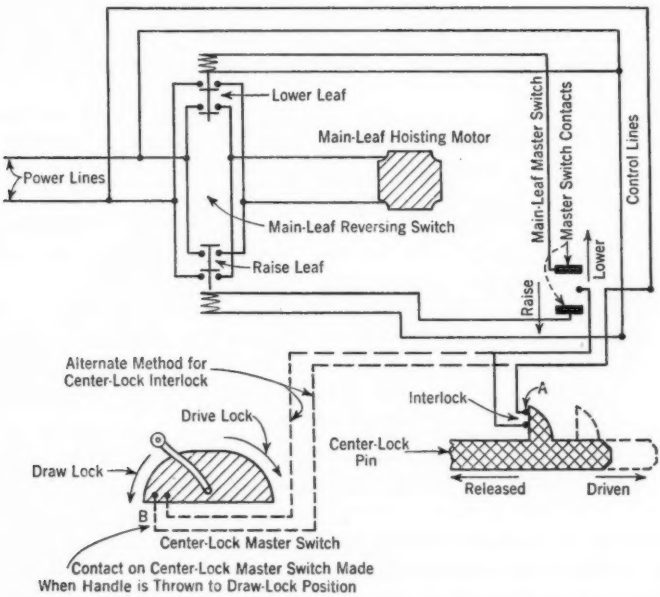

FIG. 25.—INTERLOCKING OF MAIN MOTOR AND CENTER-LOCK PIN

are led through a pair of contacts, one on the center pin and the other on its mating seat, then it is obvious that the leaves must be seated and the locking pins fully driven before the structure can be cleared for roadway traffic.

Fig. 26 is a schematic diagram showing interlocking contacts at the center lock for the barrier and gate motor circuits. If the two contacts are at A and B, it is apparent that the gate or barrier circuit can be energized whenever the pin is driven forward, regardless of the position of the leaves, in which case additional interlocking should probably be provided on either the live-load shoes or the anchor columns. On the other hand, if the contacts are located at A and C, no additional interlock appears necessary.

If the gate or barrier circuits are interlocked with the leaves but not with the tail or center lock, it is possible negligently to permit traffic over an un-

locked span, thus inducing live-load stresses not contemplated in the original design.

In general, the sequence of all of the remaining operations should be controlled by arranging each circuit so that it remains open until all of those operations, which normally should precede it, have been completed.

For low-power circuits, such as those for operating gates, signals, etc., the control switches may be cut directly into the line. For the main (high-power) operating circuits, however, magnetic contactors should be employed.

As an example of the dangers inherent in incorrect interlocking arrangements, assume that contact A in Fig. 25 were placed on the center-lock master switch instead of on the pin itself. In such a case, if the roadway gates or barriers were open, the circuit to the center-lock motor would be open and the handle on the center-lock master switch could be thrown to the draw-lock position with the pin still closed. Such an arrangement removes the safeguard

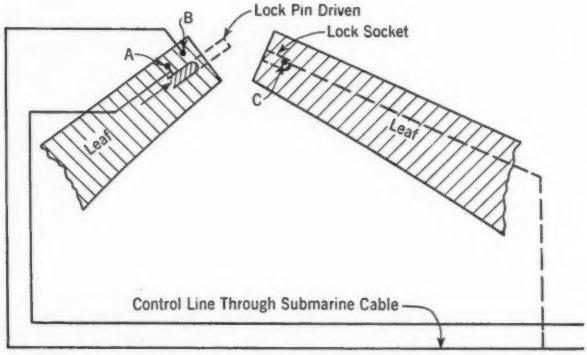

FIG. 26.—METHODS OF INTERLOCKING CIRCUIT TO BARRIER OR GATE MOTORS

of interlocking on the main hoisting motor circuit, thus making it possible for negligent operation to cause serious damage.

The foregoing are a few typical instances of electrical interlocking on bascule bridges. With the tremendous development in traffic densities and traffic speeds, the provision of adequate safeguards in this category becomes increasingly important. Each installation obviously presents its own interlocking problems. Sight distances, traffic speeds, traffic densities, etc., all operate to modify the arrangement of details. In a report covering this phase of bascule bridge design made by the writer in 1931,[11] the following general features of electrical interlocking were listed as necessary:

> "(1) Each control circuit should be so wired that the prior operation must be actually and completely performed before it becomes operative.
>
> "(2) Interlocking contacts must be positive in action and so located as to minimize the danger of short-circuiting or deterioration from moisture, ice, or mechanical injury.

[11] "Electrical Equipment on Movable Bridges," by C. B. McCullough, A. L. Gemeny, and W. R. Wickerham, *Technical Bulletin No. 265*, U.S.D.A., Washington, 1931, p. 38.

"(3) Contacts should preferably be made by a spring or snap device rather than by a sliding motion in order to avoid sparking and injury to contact points or tips, although a certain amount of wiping motion may be advantageous in tending to keep the contacts clean and bright.

"(4) Make-and-break contacts for interlocking should not be placed directly in a heavily loaded power line but rather on an auxiliary or control circuit wired to operate a magnetic contactor cut into the main power line.

"(5) Cross interlocking of operations should be provided wherever there is a danger due to the possibility of a failure of one phase of operation.

"(6) Short-circuiting buttons or switches should be provided to permit operation in the event of a failure of any portion of the interlocking mechanism.

"(7) In general, the entire interlocking arrangement should be as simple as possible, compact, sturdy, protected to the maximum possible extent, and so designed as to eliminate as far as possible every conceivable traffic hazard and every contingency or event that might expose the structure to undue stress."

In addition to the necessity for interlocking as a method of securing the correct sequence of operations, it is obviously necessary to provide devices in the nature of limit switches for controlling or limiting the movement of the leaves. Thus the wiring diagram for an electrically operated bascule structure becomes quite complicated.

It was with considerable interest that the writer noted the mention of Mr. Pihlfeldt (see "Conclusion"), who for many years was engineer of bridges for the City of Chicago. It was Thomas Pihlfeldt who took the writer on his first inspection trip over the bascule bridges of Chicago nearly a quarter of a century ago and who first impressed upon him the necessity for the bridge engineer becoming fully conversant with the principles underlying electrical operation and control.

In closing, the writer wishes again to emphasize the importance of Mr. Becker's paper and its value to engineers engaged in this line of specialized endeavor. The writer has taken the liberty of adding these few pages in regard to electrical control and interlocking for the reason that this phase was not covered by Mr. Becker and for the further reason that credit for many of these features belongs to the Chicago group, to Mr. Pihlfeldt, to Hugh E. Young, M. Am. Soc. C. E., engineer of bridge design, and to others in the city bridge engineering force. Never has a group of engineers more successfully pioneered the development of a bridge type especially adapted to local needs. In the perspective of more than forty years, the wisdom of the early type selections stands out unchallenged.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

RAINBOW ARCH BRIDGE OVER NIAGARA GORGE A SYMPOSIUM

Discussion

BY C. M. GOODRICH, AND T. KENNARD THOMSON

C. M. GOODRICH,[24] M. AM. SOC. C. E.[24a]—In the discussion of the design of the Rainbow Bridge "elastic theory" is frowned upon, and a "deflection theory" is preferred. Elastic theory deals with deflections, and is based on the law of conservation of energy, a law not lightly to be repealed or suspended. If one made a blunder in a mathematical problem, or failed to find a solution in one particular way, the remedy would be more mathematics. In the case of the arch the remedy is more elastic theory, unless indeed the elastic limit is threatened.

In suspension bridges what is here termed "the elastic theory" is based on the assumption that the stiffening truss rises and falls in a gracefully parabolic manner, which does not accord with elastic theory. It would perhaps be more nearly correct to call it the "inelastic method."

In the very common case of a boom the dead load causes a deflection and a consequent series of moments and deflections due to the axial load; often the biggest effective load on a long boom is due to the swinging and the series of moments and deflections that come from that; the writer proportions also for an inaccuracy of 1/750 the length in each of two directions. A boom 180 ft long was once used to erect the towers for a lift bridge; such a boom deserves respectful consideration. Only three times has the writer gone through the somewhat tedious operation suggested; and the experience of others no doubt will be similar. This case is clearly one of limit design, like the arch.

If "Stern's Theory"[9] contains something more than computing an arch by bits and pieces in the long-familiar way, or the investigation and cure of critical deflection, if any, it would be useful to have the procedure explained.

NOTE.—This Symposium was published in October, 1943, *Proceedings*. Discussion on this paper has appeared as follows: December, 1943, by Egidio O. Di Genova, and Charles Mackintosh.

[24] Cons. Engr., The Canadian Bridge Co., Ltd., Walkerville, Ont., Canada.

[24a] Received by the Secretary November 27, 1943.

[9] "A Deflection Theory of Two-Hinged Arch Ribs," by G. Stern, thesis No. 516, Dept. of Civ. Eng. Columbia Univ., New York, N. Y. (a résumé of theoretical studies made at the Technical Univ. of Vienna, Austria).

According to the authors of the third paper (see "The Elastic Centroid—Its Application to the Study of the Action of the Closed Rib"), the method of the elastic centroid "was presented by H. Müller-Breslau in 1925." The writer's first acquaintance with this Müller-Breslau method was shortly after 1910, when the late David A. Molitor, M. Am. Soc. C. E., published[25] a very clear and complete explanation of its use for fixed arches. His work contains also an interesting case of its use in a problem with two redundants. The elastic centroid is also called "center of instantaneous revolution," since it is determined by the moment applied at the base. A still more appropriate name is perhaps "imaginary hinge"; this is the name for like devices used in several problems by August Föppl.[26]

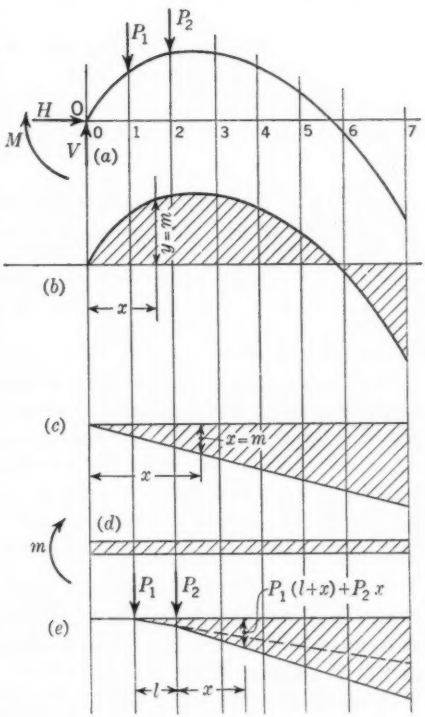

FIG. 39

Such fixed arches may be figured by a modest extension of the two area-moment theorems, announced in 1873 by the late Prof. C. E. Greene, M. Am. Soc. C. E. One need only bear in mind that in most cases $\int \frac{m\,M}{E\,I}\,ds$ may be represented conveniently by an area $\int \frac{M}{E\,I}\,ds$, when the expression means the statical moment of a series of areas about either of two axes; or, when m is a unit moment, the sum of these areas.

The arch may be divided into any number of segments n, preferably agreeing with the changes in arch section (see Fig. 39(a)). Taking moments about a horizontal axis through point O:

$$0 = -H\sum_{0}^{6} k\,A\,\bar{y} + H\,k_{6,7}\,A_{6,7}\,\bar{y} + V\sum_{0}^{7} k\,B\,\bar{y}$$

$$+ M\sum_{0}^{7} k\,l\,\bar{y} - \sum_{1}^{7} C\,k\,\bar{y} \qquad (37a)$$

in which (in addition to the Symposium notation): $k = \frac{\lambda}{l\,I}$; A and B are the

[25] "Kinetic Theory of Engineering Structures Dealing with Stresses, Deformations and Work for the Use of Students and Practitioners in Civil Engineering," by David A. Molitor, McGraw-Hill Book Co., Inc., New York, N. Y., 1911.

[26] "Vorlesungen über technische Mechanik," by August Föppl, Leipzig, 1902–1912.

m-areas in section n due to the horizontal and vertical unit dummy loadings, respectively; $\bar{y}$ = vertical distance from a horizontal line through O to that point in the arch intersected by a vertical through the centroid of the moment area with which y is used; l = length of the horizontal projection of section n; λ = axial length of section n corresponding to l; and C = the M-area in section n due to P loads. Taking moments about a vertical through point O:

$$0 = -H\sum_{0}^{6} k\,A\,\bar{x} + H\,k_{6,7}\,A_{6,7}\,\bar{x} + V\sum_{1}^{7} k\,B\,\bar{x} + M\sum_{0}^{7} k\,l\,\bar{x} - \sum_{1}^{7} C\,k\,\bar{x} \quad \ldots\ldots (37b)$$

in which $\bar{x}$ = the horizontal distance from O to a vertical through the centroid of the moment area with which it is used. Finally, summing areas:

$$0 = -H\sum_{0}^{6} k\,A + H\,k_{6,7}\,A_{6,7} + V\sum_{0}^{7} k\,B + \sum_{0}^{7} M\,k\,l - \sum_{1}^{7} C\,k \quad \ldots (37c)$$

In computing the deflection (say at point 2, Fig. 39(a)), consider the tangent at that point to be unchanged, and compute the movements of points 0 and 7 relative to point 2. For a horizontal movement at 0 relative to 2:

$$E\,\rho = -H\sum_{0}^{2} k\,A\,\bar{y} + V\sum_{0}^{2} k\,B\,\bar{y} + M\sum_{0}^{2} k\,l\,\bar{y} - C_{1,2}\,k_{1,2}\,\bar{y} \quad \ldots (38)$$

The equations for the other side would be similar except that the x-values would be measured from the right and the y-values from point 7. For the two-hinged arch, omit the M_0-terms and Eq. 37c. For temperature use $E\,\rho$ in Eq. 37a instead of 0.

Clearly the foregoing method is adapted to finding the first and second shape changes, or to checking the result of an assumption. This ignores, of course, the effects of shear, of possible variations in material and workmanship, of the yielding of steel concrete and rock in the bases, of splices, and of connecting members. Mr. Molitor considered the base as yielding. It would be interesting to know what, if any, measurable displacement occurred in the length of the Rainbow Bridge arch.

The writer would have preferred straight segments in the arch, as being cheaper and without unrelieved stresses; he believes the esthetic effect would have been equally good, due to the fact that the eye sweeps over such a figure in a curve, as shown in psychological laboratories by tracing the movements of the eyeball. A camelback with say three straight segments in the top chord is ugly; with say seven or more it is pleasing, because the eye passes over the top chord in a curve. The mass of the arch is of modest size relative to the mass of the roadway as seen at an angle; the connections between the two are too slight to effect an esthetically satisfying unity.

T. KENNARD THOMSON,[27] M. AM. SOC. C. E.[27a]—It is a pleasure to read in the first Symposium paper (heading, "Preliminary Studies") that the engineers realized that such a structure should "blend with the natural beauty and grandeur of the Niagara gorge." The Rainbow Arch Bridge is the fifth to be erected at approximately this site—probably the only case of so many renewals of an iron or steel bridge in America. The first three were undoubtedly iron bridges and the last two steel:

No. 1—A suspension bridge, 10 ft wide, was opened for traffic on January 2, 1869.

No. 2—A suspension bridge replaced No. 1, with a width of 17 ft. This bridge was finished on December 15, 1888, and blew down on January 9–10, 1889.

No. 3—No. 2 was rebuilt also as a suspension bridge, 17 ft wide, in 1889.

No. 4—A steel arch replaced No. 3, and was completed in August, 1897, and destroyed by ice on January 27, 1938.

No. 5—The Rainbow Arch Bridge, as described by the authors, was opened for traffic on November 1, 1941.

The late John A. Stewart, in 1911, acting on his own initiative, invited a group of Americans and Canadians to celebrate the 100th Anniversary of the Treaty of Ghent—100 years of peace between the two countries. He had many prominent men on his Commission, including: Joseph H. Choate, Jacob H. Schiff, Gutzon Borglum, the great sculptor, Sir Edmund Walker, president of the Canadian Bank of Commerce (who was the most outstanding Canadian banker of his time), and many others. He also honored the writer by asking him to serve and to design two bridges for the event. The first World War put an end to the proposed celebration.

One of these bridges, designed by the writer, was to be built between Buffalo, N. Y., and Fort Erie, Ontario, Canada, and the other was to replace No. 4, at the site of the Rainbow Arch Bridge. At that time bridge No. 4 was known to be inadequate and unsafe, and the thought was that a bridge built to celebrate 100 years of peace should endure for several hundred years—which no steel bridge, at this site, can ever do, unless a far better paint is developed to resist the spray from the falls, the ice, etc. The writer proposed a heavily reinforced concrete arch, with granite facing (see Fig. 40). The Buffalo Bridge, built by others, also had the same location, number of spans, and similarity of outline as the writer's plan of 1911.

When the writer spoke to Sir Edmund Walker about these proposed bridges, he said that he would not be in favor of bridges; they were too utilitarian; but, when he was shown photos of paintings of the writer's two bridges, he said at once, "Oh, that is different—they are beautiful." When Mr. Stewart showed the two paintings—one 5 ft and one 8 ft long—to Mr. Schiff, he at once told Mr. Stewart: "You cannot get up plans like those without money," and voluntarily gave Mr. Stewart his personal contribution, a check for $5,000.

[27] Cons. Engr., New York, N. Y.

[27a] Received by the Secretary December 9, 1943.

At a subsequent luncheon, attended by about forty people, one of the most prominent of the New York architects said that suitable architectural plans should be prepared (he would like to have made them). When asked what he thought such plans would cost, he replied, "About $50,000." He was then shown photos of these two paintings, and he exclaimed: "Oh, you have the plans already," and lost interest in the subject. This is a good example (many

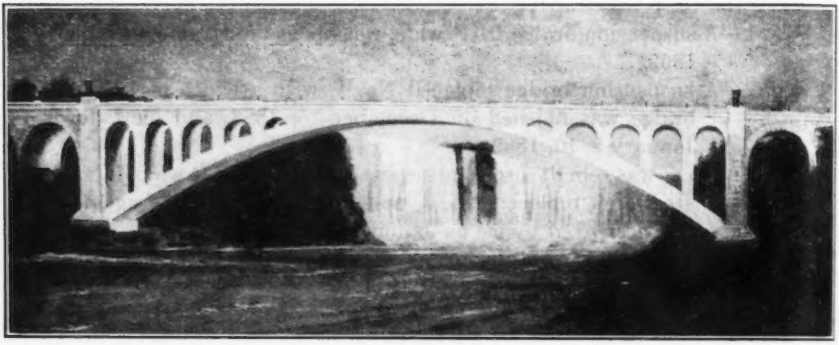

FIG. 40

others could be cited) of how men in other professions are much more adept in knowing what to charge for their services than most engineers are. It also shows that it is well worth while to have simple but beautiful outlines for all engineering structures.

Simplicity of design gives beauty without expense; whereas much intricate filigree work is very expensive and not so beautiful, as exemplified in many cathedrals.

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DISCUSSIONS

FLOW CHARACTERISTICS AT RECTANGULAR OPEN-CHANNEL JUNCTIONS

Discussion

BY EDWARD H. TAYLOR

EDWARD H. TAYLOR,[19] JUN. AM. SOC. C. E.[19a]—The interest shown in this rather specialized problem has been an agreeable surprise to the writer. The discussers have uniformly realized that the purpose of the research was to investigate the degree to which elementary physical laws can be applied to the situations devised, rather than to discuss the problem of turbulent mixing in detail. The mixing problem is essentially complicated, and thus should be approached from a consideration of pressure and momentum. If it is possible to write down correct expressions for the pressures and momenta at the boundaries of the process in question, the intermediate details can be disregarded.

Mr. Palmer suggests that the problem should be studied from the standpoint of energy loss rather than from that of change of momentum. This would require knowledge of the mechanism of energy loss, which was not contemplated in the original studies. Thus, in the absence of theoretical considerations relating to these losses, such an approach would seem to require greater recourse to empiricism than had been intended.

In raising the question of evaluation of the pressure force on the walls of the branch channel, both Mr. Hickox and Mr. Stevens have accurately stated the chief source of disparity between the experimental results and the equations. The measurements substantiated the assumption of equal depths above the junction; but this, of course, disregards wall pressures arising due to curvature of flow in the branch between the junction and the point of measurement of depth. The writer now recognizes that failure to measure the static pressures in the region constitutes an important omission.

NOTE.—This paper by Edward H. Taylor, Jun. Am. Soc. C. E., was published in November, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1943, by Harold K. Palmer; September, 1943, by Messrs. G. H. Hickox, J. C. Stevens, and C. J. Posey; and October, 1943, by Messrs. I. B. Hosig, L. Standish Hall, and Brooks T. Morris.

[19] Washington, D. C.; formerly Instr., Mech. Eng., Univ. of California, Berkeley, Calif.

[19a] Received by the Secretary December 8, 1943.

Mr. Stevens brings up the same question in connection with pipe intersections. In the article he cites,[8] the pressures are assumed uniform over the pipe cross sections immediately upstream from the junction. In measurements[20] made at the University of California on a 90° pipe intersection, this factor was investigated. It was found that an appreciable lateral pressure gradient existed in the branch for some distance upstream. This is due to the fact that the flow in the branch does not change abruptly on reaching the junction, but starts to curve while it is yet in the straight part of the branch. This section of the flow system is included in writing the equations, since at its upstream end the direction of the velocity is known to be parallel to the walls. However, its inclusion introduces a component of pressure force in the direction of summation which cannot be accounted for by elementary methods.

Professor Posey correctly states that no consideration has been given to cases involving flow at less than critical depth. The same fundamental principles would apply to such cases, although the problem of maintaining supercritical velocities throughout the junction might prove troublesome. It is also likely that the factors causing disagreement would be amplified by the higher velocities.

The comments of Mr. Hosig regarding the practical aspects of this research are very welcome, since such considerations were quite beyond the scope of the original studies. It is gratifying to have Mr. Hosig's testimony that these results are not entirely academic.

In his examination of the momentum equation, Mr. Hall has reached certain conclusions which are not correct, and at the risk of appearing pedantic, the writer wishes to point out the errors involved. Mr. Hall finds that "under the condition of no flow in the side channel, the velocity head is always one half of the depth." The algebraic steps leading to this result involved division by zero, which, of course, invalidates any subsequent conclusion.

In Eq. 9, Mr. Hall has derived the hydraulic jump relationship, but appears concerned because of the possibility that the jump might occur in the reverse direction, which he notes, is "contrary to accepted hydraulic theory." Mr. Hall has overlooked the fact that, in so far as momentum changes alone are concerned, the jump is quite a reversible process. A reverse jump would require an increase in total energy in the direction of flow, which presumably accounts for the rarity of its occurrence in nature.

Both Mr. Morris and Mr. Stevens note that the momentum terms are based on average velocities, and, to be strictly correct, such terms should contain a velocity distribution coefficient. Although this is undeniably true, it seems to be of secondary importance in comparison with inability to express all pressure forces correctly. This is particularly true of the experiments described in this paper, in which tranquil flows were maintained in all cases. Mr. Morris draws attention to Mr. Bickerstaff's photographs, which are isolated examples taken from a large collection of very good pictures. In each

[8] "Theoretical Energy Losses in Intersecting Pipes," by J. C. Stevens, *Engineering News-Record*, July 22, 1926, p. 140.

[20] "Energy Loss in Mixing Streams in Closed Circular Conduits," by William Reeves Shuler, thesis presented to the Univ. of California in Berkeley in May, 1940, in partial fulfilment of the requirements for the degree of Master of Science (unpublished).

case the dye had been injected so that, by the time it had reached the junction, it occupied the full depth of the section. If the writer's memory serves correctly, the interface remained essentially vertical in the vicinity of the intersection.

Messrs. Hickox, Stevens, and Morris are in substantial agreement on the uncertainties in the analysis as presented. In the event that a problem similar to this one should be investigated in the future, the experimenter would be well advised to heed their comments.

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DISCUSSIONS

ORIGINAL AND READJUSTED REPAYMENT CONTRACTS, BOULDER CANYON PROJECT

Discussion

BY W. L. CHADWICK

W. L. CHADWICK,[5] M. AM. SOC. C. E.[5a]—A full and interesting description of the power contracts for the Boulder Canyon Project is presented in this paper, which contains valuable information on the reasons for many of the paragraphs and clauses. As the author states, the problem of allocating the cost of the multiple-purpose Boulder Canyon Project to the several beneficiaries was encountered in only an unimportant way because the Boulder Canyon Project Act required, before money for construction could be appropriated, that provision be made by contract for revenues sufficient to insure payment of all expenses of operation and maintenance and repayment of construction advances by the United States, with interest at 4%. Since the rate for firm energy was determined by the competitive costs of steam generated energy at load centers in Southern California, there was no problem of allocating costs for rate-fixing purposes. Furthermore, the flood control allocation of $25,000,000 was statutory. Although a price was set for water storage service, this rate was not based on capital investment allocated to the purpose. Allocation complexities, however, did enter importantly into the drafting of the numerous contracts under the Boulder Canyon Project Adjustment Act.

Under the original contracts the "power privilege" was leased to the City of Los Angeles and Southern California Edison Company. Under this lease the city undertook to generate the energy requirements of the Metropolitan Water District of Southern California; the municipalities of Pasadena, Burbank, and Glendale; the states; and itself. The company undertook to generate the energy required by the two other companies and itself. The United States agreed to pay the lessees "in the form of credits upon the account of

NOTE.—This paper by Clay C. Elder, was published in December, 1943, *Proceedings*.

[5] Chf. Civ. Engr., Southern California Edison Co., Los Angeles, Calif.

[5a] Received by the Secretary December 30, 1943.

such lessee * * * the cost incurred by it in generating energy for other allottees * * *" and agreed to "require such other allottees to repay such cost to the United States."

The lease contract included the obligations of the lessees to take and pay (pay, in any case) for the energy allotted to each. Separate contracts were then executed by the other allottees requiring, similarly, that each take or pay for the energy allotted.

As the author states, conditions during 1936 and 1937 indicated that the price of 1.63 mills per kw-hr established for falling water would considerably more than liquidate the project costs. The late F. E. Weymouth, M. Am. Soc. C. E., then general manager and chief engineer of the Metropolitan Water District, realizing that the taxpayers and ratepayers of the district would be required to contribute 36% of all revenues collected in excess of actual costs, began negotiations for a downward adjustment of rates. The district was particularly interested, because the economic depression had made large inroads into anticipated water sales and it was apparent that foresight in contracting in advance for a pumping power supply that would be adequate for many years would require payment over a long period for large quantities of energy which could not be used and for which there then seemed little probability of making effective temporary releases for salvage sales.

The first efforts were directed toward obtaining a revision of rates under the lease and energy contracts then in effect. Meeting opposition from those interested in excess revenues under the original contracts, Mr. Weymouth proposed revised contracts based on the repayment of all costs only, rather than on the requirement that charges be fixed by competitive conditions. Although long and tedious negotiations were required before this principle was enacted in legislation, it is the basis for rates under the Adjustment Act.

For reasons not known to the writer, during preliminary conferences and negotiations on the adjustment legislation, it was proposed that the lease of power privilege be canceled and that the United States operate the generating plant, as well as the dam, penstock, outlets, town site, utilities, etc. Alternatively, it was proposed that generation be by means of agency agreements between the United States and the lessees. Legal difficulties involving security of tenure and loss of the valuable leasehold rights of both lessees were resolved by a form of agency contract with about the same tenure to the lessees as previously enjoyed under the lease. Under the agency agreements, the agents agree to generate energy under substantially the same bases for the same allottees as formerly.

Under the original agreements, the rate of 1.63 mills per kw-hr covered the use of falling water delivered to the butterfly valve on the upstream side of the turbines. Accordingly, associated with this rate were costs for construction, operation, and maintenance of the dam; penstocks; outlet structures; power plant building; common station facilities, such as cranes, elevators, and offices; and Boulder City, with its utilities, parks, streets, and administration; etc. However, the rate did not include costs for investment in, or operation, maintenance, and replacement of, turbines, generators, switchgear, switch racks,

power conductors, and related equipment. The latter were costs repaid by the lessees and collected, in turn, from the allottees by the United States.

Under the Adjustment Act it was required that:

> "The Secretary of the Interior is authorized and directed to, and he shall, promulgate charges, or the basis of computation thereof, for electric energy generated at Boulder Dam during the period beginning June 1, 1937, and ending May 31, 1987, computed to be sufficient, together with other net revenues from the project, to accomplish the following purposes:
>
> "(a) To meet the cost of operation and maintenance and to provide for replacements * * *.
>
> "(b) To repay to the Treasury, with interest at 3 per cent, the advances to the Colorado River Dam Fund, made prior to June 1, 1937, within fifty years from that date.
>
> "(c) To provide $600,000 for each of the years [payable $300,000 each to States of Arizona and Nevada] * * *.
>
> "(d) To provide $500,000 for each of the years * * *." (To the Colorado River Development Fund for additional development in the Colorado River Basin.)

The Adjustment Act required no adherence to the original principle of dividing the rate into a falling water charge and charges for generation. Accordingly, the entire subject of rates and "rate architecture," as referred to by W. F. Durand, was opened during the hearings held in Los Angeles during 1940 with R. V. L. Wright presiding as special representative of the Secretary of the Interior. Mr. Durand and Leland Olds served as special advisers. An early principle agreed to, however, was that the revisions in cost allocations or in rate design should not change the obligations of any allottee at the expense of another.

Allocations of cost for rate purposes were complicated by the size of generating units, the design of the generating plant, the number of allottees, and differences in standards of service on the several power systems involved.

Boulder Canyon power plant (January, 1944) contains six generators on the Nevada side and six on the Arizona side, all of 82,500-kva capacity except for one of 40,000 kva on the Arizona side (unit A-8). The distribution of these machines between generating agents and the allottees for whose benefit the machine is operated is as follows:

Generating units	Allottee for whose benefit the machine is operated
Nevada Side—	
N-1	City of Los Angeles
N-2	Burbank
N-3	Glendale
N-4	Pasadena, Boulder City, California Pacific Utility Company, and Citizens Utility Company (contractors for the Metropolitan Water District), and the State of Nevada for the Southern Nevada Power Company and the Lincoln County Power District No. 1
N-5	Metropolitan Water District and Basic Magnesium, Inc.
N-6	Metropolitan Water District and Basic Magnesium, Inc.

Arizona Side—

A-1.....City of Los Angeles et al. (see N-1 to N-4)
A-2.....City of Los Angeles et al. (see N-1 to N-4)
A-5.....Southern California Edison Company
A-6.....Southern California Edison Company
A-7.....Southern California Edison Company
A-8.....California Electric Power Company

The Southern California Edison Company is the agent operating units A-5, A-6, A-7, and A-8. All the remainder are operated by the City of Los Angeles.

Including three contractors for energy allotted to but not used by the Metropolitan Water District, there are thirteen agencies receiving energy directly from the Boulder plant. Transmission to City of Los Angeles is at 287,000 v; to Edison Company at 220,000 v; to Metropolitan Water District and Basic Magnesium, Inc., at 220,000 v; to California Electric Power Company and Lincoln County Power District at 138,000 v; to Burbank, Glendale, and Pasadena via City of Los Angeles lines at 287,000 v; to Southern Nevada Power Company at 33,000 and 69,000 v; to Citizens Utilities at 69,000 v; to California Pacific Utilities Company at 69,000 v; and to the United States at 33,000 v. Further complicating the situation are the different frequencies. At the time construction of the plant was undertaken, all Southern California utilities, except California Electric Power Company (then Southern Sierras Power Company) and Los Angeles Gas and Electric Company (later part of the City of Los Angeles system) were operated at 50 cycles. In 1936 and 1937, the City of Los Angeles converted its system to 60 cycles, which, because of joint transmission obligations, required similar conversion by Burbank, Glendale, and Pasadena. Accordingly, the Southern California Edison generators are operated at 50 cycles and all others at 60 cycles. Hence, parallel operation of all units is not possible. Such parallel operation is further undesirable because of the tremendous energy concentrations to be interrupted during short circuits.

Accordingly, after determining the costs to be met (that is, for investment, interest, payments to states, Colorado River Development Fund, operation, maintenance, and replacement) equitable allocation of shares in these costs between allottees and contractors was necessary. At the request of the special board, most of the allottees submitted suggestions for rate design. These varied from complicated bases involving consideration of all differences between allottees to others which would have allocated all on a straight kilowatt-hour allocation. The complications were decidedly undesirable, and the straight kilowatt-hour charge was inequitable because of differences in load factor, cost of generating equipment, differences in operating costs, etc.

An equivalent of the original falling water charge was determined by estimating all costs and charges for the 50-yr contract period, estimating the generation of both firm and secondary energy during the same period, and determining the quotient of the two.

The energy component was finally resolved by dividing the generation and transforming and switching facilities into sections according to uses, determining the investment costs for each, estimating the operation, maintenance, and replacement costs for a year in advance, and billing monthly for the total of such costs per section. Adjustments are made at the end of each operating year.

Costs for facilities used for joint service, such as generators N-1, N-2, N-3, and N-4, were finally allocated on the basis of kilowatt-hour use, although much testimony was taken favoring a kilowatt basis for this purpose.

Essentially, with two minor exceptions, it may be said that

(1) Allocation of all common costs for the dam and appurtenances, general plant, and other facilities necessary to all generation is entirely on a delivered kilowatt-hour basis;
(2) Allocation of costs for generation is on a direct recorded cost basis; and
(3) Allocation for jointly used generating facilities is on a kilowatt-hour basis.

The exceptions are that the cost of common station facilities operated by agents is allocated on the basis of generator months and that operation costs of Southern California Edison Company for operating its own generators and those of the California Electric Power Company are allocated on the basis of generator units operated.

Because billings must be made from month to month, they are based on estimates made a year in advance, adjusted at the end of each year.

Realizing the impossibility of estimating costs and energy output accurately, fifty years in advance, the Adjustment Act contract and regulations provide for adjustments in energy rates at specified times.

The Adjustment Act contracts have now (1944) been in effect for nearly three years. Although, like all negotiated contracts, those under the Adjustment Act involve numerous compromises (based on neither pure economics nor engineering), no fault has yet developed in the allocation bases to indicate their impracticability.